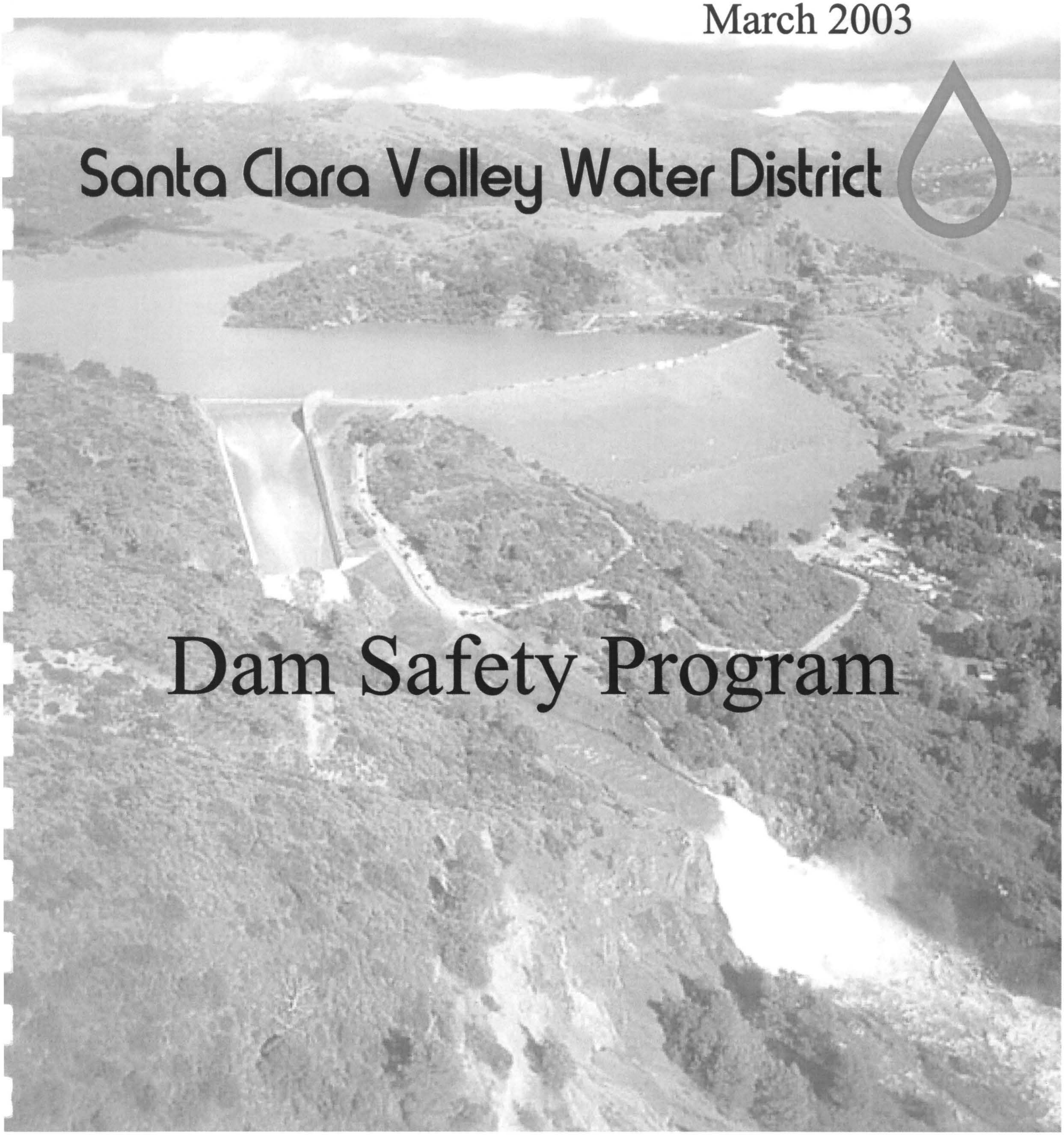
March 2003

Santa Clara Valley Water District



Dam Safety Program

Santa Clara Valley Water District

MEMORANDUM

TO: Dave Hook, P.E.
Engineering Unit Manager

FROM: Richard L. Volpe, P.E., G.E.
Senior Engineer

SUBJECT: Dam Safety Program
Transmittal of Final Draft Report

DATE: March 26, 2003

The enclosed report presents the results of an extensive two-year effort to develop a new Dam Safety Program (DSP) exclusively for the Santa Clara Valley Water District. The DSP as described in the report consists of two separate and distinct elements. The first element is a recommended set of methodologies, procedures, and inspections that should be followed by current and future District staff to ensure that all District dams are being safely operated and maintained to ensure the public safety. The second element includes a one-time effort, referred to as the Dam Instrumentation Project (DIP). This is a major project to plan, design, install, and automate a state-of-the-art geotechnical instrumentation program at ten District dams.

The report consists of four chapters and nine appendices. The four chapters of the report focus on the following: Chapter 1 presents a brief summary of the history of the District, describes how and why the dams were constructed and the need for a formal dam safety program at the District; Chapter 2 presents a brief history of dam construction and dam safety issues in the United States; Chapter 3 presents a detailed discussion of the elements comprising the newly proposed DSP; and Chapter 4 presents the staffing requirements, schedule and estimated cost of implementing the DSP. The nine appendices provide reference information dealing with the eleven District dams, including: dam statistics; cultural and construction history; geologic and seismic issues; a post-earthquake inspection program; geotechnical engineering issues and analyses; instrumentation design issues; automated data acquisition systems; preliminary instrumentation design; and a list of technical references. A major portion of the appendices represents a unique one time effort to provide historical documentation of historical data, observations, and events that have been developed, primarily by the report's principal authors, over the past 30 years of consulting to the District. Although the information has immediate application, the appendices are primarily intended to act as a principal reference for future engineers and geologists who will assume responsibility for dam safety here at the District.

The enclosed report is considered a final draft and a presentation by its principal author will be given to Senior Management on April 2, 2003. The report will then be submitted to the California Department of Water Resources, Division of Safety of Dams, for its peer review and comment. Following completion of the internal and DSOD reviews, the final report will be prepared and distributed. It is estimated that the final report should be available by June, 2003.

Richard L. Volpe

Richard L. Volpe, G.E.
Senior Engineer – Dam Safety Section
Water Utility Engineering Unit

Cc (w/attachment): W. Wadlow, A. Bennetti, R. Yep, B. Fields, R. Whipp, J. Nelson

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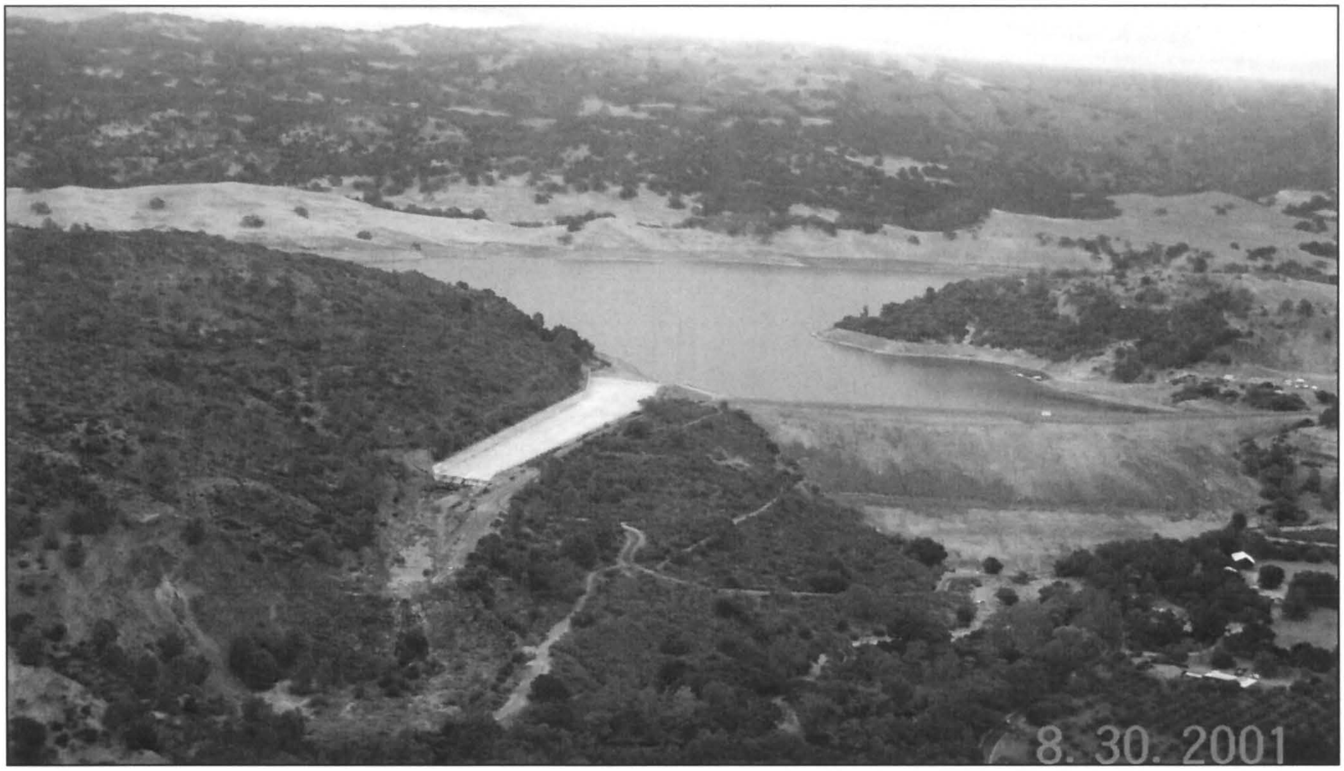
Santa Clara Valley Water District

Dam Safety Program



8. 30. 2001

Prepared by:

Water Utility Operations Division

Water Utility Engineering Unit

Richard L. Volpe, P.E., G.E.

James L. Nelson, C.E.G.

March 2003

Executive Summary

The District has a 67-year-long proud history of successfully operating and maintaining its dams. During this period, the dams have been maintained in continuous operation serving the water needs of Santa Clara County. During this period, there has not been one incident involving dam safety or damaging release of water from any of the District dams. Much of the credit for this stellar District record goes to employees who have since retired, and to previous and current members of our Board of Directors whose actions have clearly shown how much they value and treasure the District's dams and reservoirs. Their legacy of excellence lives on and will be memorialized with the formal adoption of this Dam Safety Program.

In 1998, an emergency was declared at Lenihan Dam due to the discovery of unexplained high water pressures that were found adjacent to the outlet pipe. This emergency might not have been necessary or warranted if the dam had been properly instrumented beforehand, as it is today. The Lenihan Dam incident was very helpful in pointing out some problems associated with our then current approach to dam safety, not the least of which was the fact that none of our dams met the current state-of-practice in the area of dam instrumentation. This awareness caused the District to take a much closer look at its dam safety efforts.

The principal author of this report was hired by the District in April 2000 to focus his efforts in the area of dam safety. The preparation of a formal report to set forth the requirements of a formal Dam Safety Program here at the District began in June 2001 with the formation of a Dam Safety Section in the reorganized Water Utility Engineering Unit of the Water Utility Operations Division. Although this report represents the 40-year professional experience in the area of dam engineering of its principal author, it has been peer reviewed by others with extensive dam safety experience in order to ensure that the procedures, methods, and proposed instrumentation design elements discussed in the report are appropriate and consistent with current engineering practice.

Unlike a report describing the design, for example, of a specific dam or a water treatment facility, this report is intended to act as a reference document for both current and future District employees involved with dam safety and other aspects related to our dams. The report documents the make-up and interrelationship of the many different elements that constitute a modern dam safety program. In fact, most of the elements associated with such a safety program are already being carried out to some extent by the District; however, an understanding of the importance of each of these elements may not be fully appreciated by the personnel to whom they are assigned.

A special discussion is presented (Section 3.5) regarding the environmental and CEQA concerns that are currently impacting the Dam Safety Program. The District is entering into new and radically different environmental oversight requirements, specifically regarding dam maintenance issues, than existed just two years ago. The significance of this internal District oversight is that it has a direct impact on dam safety. Based on safety inspections completed in 2002, and subsequent discussions with the DSOD inspector, it is possible that future restrictions of our reservoir operations could be imposed by DSOD as a result of delays in completing required maintenance work. As a result of these concerns, it is recommended (Section 3.6) that the management and oversight of the dam maintenance program be transferred for the next 4-5 year period to the Dam Safety Section of the Water Utility Engineering Unit. One of the principal objectives of such a position would be to work very closely with the Watershed Planning Unit in the development and completion of a Programmatic EIR for dam maintenance at our dams that would be similar to the effort that was recently completed for our stream maintenance. This will require the addition of at least one fulltime staff position that would be responsible to track and oversee maintenance and to assist and facilitate environmental planning of dam maintenance issues. After the initial 4-5 year effort to implement the Programmatic EIR, it may be desirable to shift the maintenance responsibility back to the Utility Maintenance Unit where the maintenance responsibility currently resides.

A significant level of effort in this report was devoted to documenting the cultural and construction history of our dams (Appendix B). A similar effort was expended in summarizing the special aspects of the geologic and seismic (Appendix C) and geotechnical engineering (Appendix E) aspects associated with the dam safety program. The summaries of these disciplines and discussions will hopefully assist and give guidance to future engineers and geologists associated with the dam safety program.

The District dams are particularly vulnerable to earthquake loading given the seismic setting within the south Bay Area. A major effort of this report was the development of a unique post-earthquake inspection program (Appendix D) for our dams that has evolved here at the District over the past 10 years. Although the basic elements of the post-earthquake inspection program are already in place and functional, this report provides the first written description of its functional description of the program and requirements. This report will form the basis for the formal implementation of the program under the Dam Safety Program. The Post-Earthquake Inspection program will be administered under the overall direction of the Dam Safety Section of the WUE Unit.

The implementation of the Dam Instrumentation Project (DIP) is at the heart of the Dam Safety Program. The DIP will include the installation of over 150 new geotechnical instruments at ten dams that require additional instruments to upgrade them to current dam safety standards. The preliminary instrumentation design for each dam was completed as a part of this report effort and is presented in detail in Appendix H. It should be noted that the overall approach to the selection of instrument types, the number and location of instruments, and the type of automation concepts, is currently being reviewed by an independent consultant (GEI Consultants). The final number of instruments therefore could vary slightly from those discussed herein. The final DIP will incorporate the review comments provided by GEI, as well as DSOD.

The estimated cost for the instruments and the automation equipment for all ten dams is estimated to be about $3.5 million with an additional $1.85 million to for final design plans and specifications plus construction management. The original plan was to take about 5-7 years to complete the DIP, thus spreading out the overall cost and providing the WUE staff time to adjust to the collection of the new data. Because of the significant risk associated with not having all of our dams sufficiently instrumented, management could elect to speed up the DIP in order to complete the total effort in say 2-3 years. As the DIP is implemented more staff will be required for the Water Utility Engineering Unit to monitor, interpret, and summarize the data collected. Once completed, it is estimated that in excess of 1,000,000 bits of data will be generated per year from the instruments. It is estimated that the Dam Safety Section within the WUE Unit will increase from its current staff of 2.5 plus engineering unit manager to a staff of 5 plus engineering unit manager during the next 5-6 year period when the DIP becomes operational..

The completion of the DSP presented in this report represents a major advancement for the District. To our knowledge, the report represents the first of its kind undertaken by a public or private dam owner in the United States. It describes a dam safety program that, if vigorously followed, should ensure the safe operation and maintenance of the District's current 11 dams for the foreseeable future. The DIP, if implemented as currently envisioned, should adequately serve the District's needs for dam monitoring for at least the next two or three decades, if not longer.

DAM SAFETY PROGRAM REPORT

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Chapter 1

Introduction

1.0 INTRODUCTION

1.1 Background and Need for Dam Safety Program

This report describes the Dam Safety Program (DSP) that has been developed specifically for the Santa Clara Valley Water District (District), a special utility district originally formed in 1929. The District owns 10 dams and two water retention facilities that are operated under licenses granted by the State of California Department of Water Resources, Division of Safety of Dams (DSOD), and the Federal Energy Regulatory Commission (FERC). As part of these license requirements, the District must provide assurances that its dams and other water retention facilities are inspected for safety on a regular basis, and that the reservoirs are being operated in a safe and prudent manner.

One of the principal justifications for writing this report is directly related to the significant risk of damage associated with the potential for dam failure here at the District. For Anderson, Lenihan, Coyote, and Stevens Creek Dams in particular, in the event of a sudden and unforeseen cataclysmic failure under full reservoir conditions at any of these facilities, significant loss of life and property damage would occur. The monetary value of potential property damage alone for each of these facilities is unknown and speculative. The DSP has been specifically designed to addresses measures to significantly improve our ability to better understand the real-time performance of our dams and thus significantly reduce this risk.

To our knowledge, this report represents a unique effort and perhaps is one of the first of its kind in the United States. Many other private dam owners have devoted major efforts to ensure that their dams are operated and maintained in a safe manner. Many privately owned dams are also being fitted with modern geotechnical instrumentation to aid in the safety monitoring. The Dam Instrumentation Project (DIP), which is a major part of the DSP, is a bold and innovative effort to instrument and fully automate all District dams and then to tie the instrumentation monitoring into one seamless program. When finished by approximately 2010, the DIP will provide the District with the assurance that all of its dams have been instrumented to the current state-of-practice and that a program is in place to successfully monitor the overall performance of each dam to meet current safety standards for well into the next several decades.

The concept for developing an updated dam safety program at the District was originally conceived and discussed following an emergency that occurred at its Lenihan Dam in 1998. District management recognized the need to have a staff of geotechnical engineers and engineering geologists, with specialized experience in the design and analysis of dams, whose primary job functions would be dedicated to dam safety. In July 2001, following the hire or reassignment of necessary personnel, a Dam Safety Section was formed within the Water Utility Engineering Unit of the Water Utility Operations Division. Two major goals for the newly formed group were identified:

- A report would be prepared to define and describe the major elements comprising an up-to-date dam safety program and how such a program should be implemented and carried forward at the District. The report would identify staffing requirements over the next five years. The Dam Safety Program (DSP) report would also document the history of construction, performance, previous instrumentation, and technical analyses of the District's 12 dams, in an effort to act as a historical reference for future staff in administering the program for the lifetime of the dams.

- The second major goal of the DSP was to complete the preliminary design of a state-of-the art automated set of geotechnical instruments at ten of the District's dam in order to monitor dam performance with time. This effort, identified as the Dam Instrumentation Project (DIP), was intended to focus the design effort on nine District dams and a water retention facility which lacked adequate instrumentation, and were therefore vulnerable to future emergencies. Lenihan Dam was excluded from the list since it had already been adequately instrumented following the 1998 emergency. A report describing the preliminary design of such an instrumentation effort was completed in January 2002 and submitted to DSOD for their review and concurrence.

This report reflects the District's commitment to dam safety. It is intended to provide a reference document for all current and future District employees involved with the operation, maintenance, and safety inspection of all existing or future dam facilities. The preliminary designs for new instrumentation at 10 of the 11 existing dams, along with methods for automatically retrieving the collected data, are also presented. These instrumentation designs are intended to provide the District with the necessary information to safely operate its dams and reservoirs for the next several decades. The current dams owned and operated by the District represent an important investment for the future reliability of Santa Clara County's water supply.

The construction and operation of the dams owned and operated by the District forms a part of the rich history of the Santa Clara Valley. The combined total of 169,415 acre-feet of potential storage provided by District dams is an integral and critical part of the diverse water resources for the area. The following sections of this introductory chapter briefly describe some of the historical background of the District. A map of the general area served by the District is presented in Figure 1-1, along with a distinction and location of the physical watershed areas, treatment plants and pump stations. A more detailed map showing the locations of current District dams is presented in Figure 1-2.

1.2 Historical Background

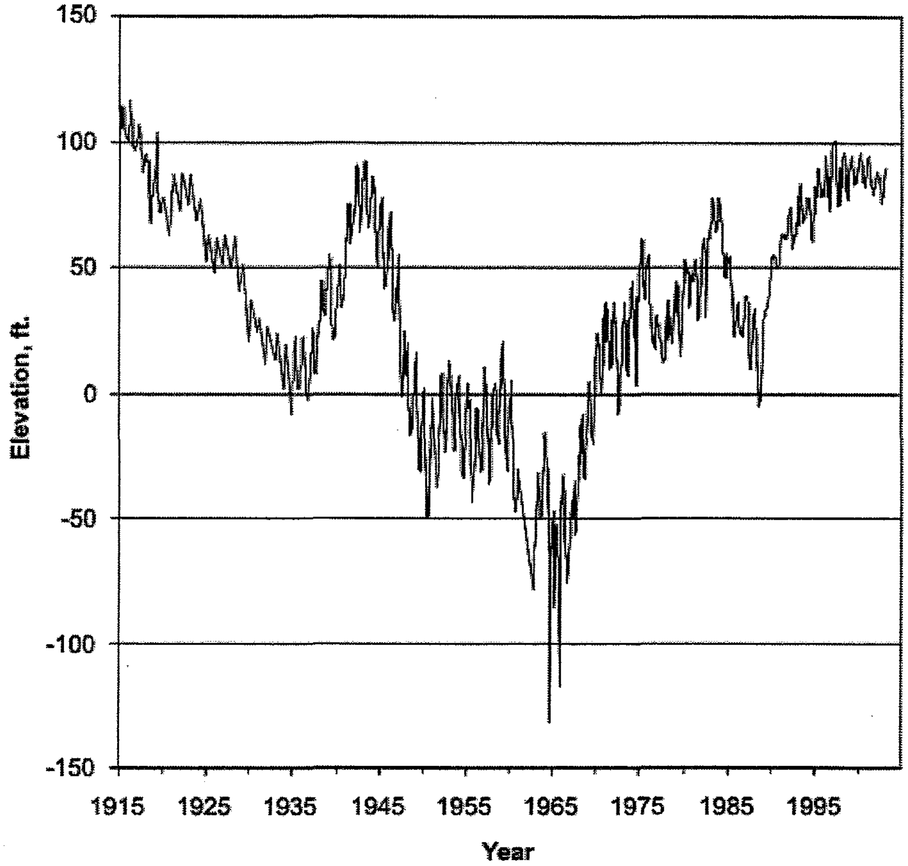
Santa Clara Valley has a rich and diverse cultural and agricultural heritage dating back over two centuries. Some of this history is captured in Appendix B which presents a discussion of the cultural history associated with lands surrounding each of our dams. As the number of acres of rich valley soil under cultivation began to increase, so too did the demand for water. By the end of the early 1900s, farmers and other water users began a more aggressive utilization of the plentiful groundwater available at that time. Prior to this time, the groundwater throughout major portions of the valley was actually "flowing" artesian when penetrated by wells. This relatively rare source of abundant water was available because the valley, along its higher margins, is ringed with outcroppings of pervious gravel deposits. These gravel deposits in turn are directly connected to, and form an integral part of, the aquifers underlying the central portion of the valley. The artesian conditions simply reflected the historical balance of surface and groundwater availability. By the early 1920s, with the ever-increasing demand for more water for both agricultural and domestic purposes, the installation of more wells, and increasing rates of groundwater extraction, overdrafting of the groundwater basin developed and continued uninterrupted for more than four decades. Some very important relief was provided in the early 1930s with the formation of the Santa Clara Valley Water Conservation District and its conservation, recharge, and dam building programs.

1.3 Origins and Mission of District

In 1921, a committee of concerned citizens commissioned a study and report on the problems of groundwater basin overdraft by Fred H. Tibbetts, a prominent civil engineer from nearby Campbell, and his partner Stephen Kieffer. Later in the same year, the resulting report, commonly referred to as the "Tibbetts and Kieffer Report," proposed a comprehensive conservation plan that included a system of dams and facilities for collecting and distributing water throughout the valley in a controlled manner for groundwater recharge.

In November 1929, after several previous failed attempts, the voters of Santa Clara County approved a ballot measure creating the Santa Clara Valley Water Conservation District (SCVWCD) under the provisions of the Water Conservation (Jones) Act of 1929, which passed in the same election. The District's original mission was water conservation, driven by the need to alleviate the problems of dropping water levels and land surface subsidence, which was becoming more and more evident and resulted from the growing number of wells pumping greater quantities of water from the valley aquifer. The original plan called for six water conservation dams to be constructed in the hills surrounding Santa Clara Valley to capture rain season flows. The captured water would then be discharged under more constant flow conditions to numerous small infiltration ponds cut into the gravel outcrops surrounding the valley for recharge of aquifers and wells during the dry summer and fall seasons.

The original six dams, including Almaden, Calero, Coyote, Guadalupe, Stevens Creek, and Vasona, were all built in 1935/36 and their reservoirs very quickly filled. The system worked according to plan until the late 1940s when the ever-increasing demand for water outstripped the valley's groundwater resources. Groundwater levels decreased and subsidence once again resumed unchecked. Subsidence of the valley floor was finally stopped in the late 1960s, save for a small amount which occurred during the 1976-77 drought when groundwater pumping increased. A plot of the historic groundwater elevation from one characteristic well in San Jose is presented in Figure 1-3. It is a plot of the monthly water level in the well from 1915 through the present and represents the longest continuous record of water level in our database. It will be noted that the District has been successful in restoring the groundwater levels to nearly the conditions that predated the subsidence nearly 100 years ago.



This plot depicts the water surface elevation of Santa Clara Valley Subbasin Index well (07S01E07R013).

Figure 1-3 Typical index well for Santa Clara Valley

1.4 District Boundaries and Groundwater Subbasins

The District included all lands in the valley floor within these boundaries that are below an elevation of 200 feet. The District's boundaries essentially followed three groundwater subbasins that underlie the San Clara Valley. The groundwater basin is divided into three interconnected subbasins that transmit, filter, and store water. The Santa Clara Valley Subbasin in the northern part of the County extends from Coyote Narrows at Metcalf road to the County's northern boundary. The Diablo Range bounds it on the east and the Santa Cruz Mountains on the west. These two ranges converge at the Coyote Narrows to form the southern limits of the subbasin. The Santa Clara Valley Subbasin is approximately 22 miles long and 15 miles wide, with a surface area of 225 square miles. A confined zone within the northern areas of the subbasin is overlaid with a series of clay layers resulting in a low permeability zone. The southern area is the unconfined zone, or forebay, where the clay layer does not restrict recharge.

The Coyote Subbasin extends from Metcalf Road south to Cochran Road, where it joins the Llagas Subbasin at a groundwater divide. The Coyote Subbasin is approximately 7 miles long and 2 miles wide and has a surface area of approximately 15 square miles. The subbasin is generally unconfined and has no thick clay layers. This subbasin generally drains into the Santa Clara Valley Subbasin.

The Llagas Subbasin extends from Cochran Road, near Morgan Hill, south to the County's southern boundary. It is connected to the Bolsa Subbasin of the Hollister Basin and bounded on the south by the Pajaro River (the Santa Clara - San Benito County line). The Llagas Subbasin is approximately 15 miles long, 3 miles wide along its northern boundary, and 6 miles wide along the Pajaro River. A series of interbedded clay layers, which extends north from the Pajaro River, divides this subbasin into confined and forebay zones.

The three subbasins serve multiple functions. They transmit water through the gravelly alluvial fans of streams into the deeper confined aquifer of the central part of the valley. They filter water, making it suitable for drinking and for municipal, industrial, and agricultural uses. They also have vast storage capacity, together supplying as much as half of the annual water needs of the County. In 2000, the groundwater basin supplied 165,000 acre-feet of the total water use of 390,000 acre-feet.

1.5 Early Years of the District

By the time the District had been formed in 1929, the piezometric head of the formerly artesian groundwater within the valley had been drawn down to between 110 and 120 feet below ground surface and the total subsidence since original ground surface measurements were started in 1912 was about 3.1 feet. Fortunately, the initial water conservation measures were effective in producing a major recovery and by the early 1940s the maximum annual depth of the piezometric surface had rebounded to less than 40 feet. Nevertheless, during the intervening years since the formation of the District and the construction of the original six dams, subsidence continued. By the time the groundwater levels had been stabilized in the early 1940s, the total subsidence from 1912 amounted to about 4.4 feet.

Following World War II, the population of the Santa Clara Valley began to increase dramatically. The population in the county jumped from 30,000 in 1940 to 90,000 in 1948, then to 291,000 in 1950. This explosive post-war growth, combined with a major drought from 1940 to 1946, put a severe strain on local water resources. Following the original success of the original six dams in producing a complete rebound by the early 1940s, the groundwater levels again continued to drop due to increased agriculture, industry and residential construction, and land subsidence worsened due to over-pumping. In response, voters passed bonds to construct two more large dams for water storage and percolation: Lexington and Anderson. During the 25-year period from 1940 to 1965, the population increased from about 90,000 to more than 650,000. During this same period, the maximum pumping lift for water increased from less than 40 feet to about 240 feet.

By 1965 it was clear that the subsidence of the valley would continue unabated unless something was done to abate the tremendous pressures on the groundwater basin due to excess pumping. This was truly a watershed year since it was when deliveries of South Bay Aqueduct imported water began (completed by a joint effort of state and federal agencies). The imported water allowed the District to begin an increased recharge program that was ultimately effective in controlling and stabilizing the valley's groundwater levels and halting land subsidence in 1967. The maximum subsidence in the valley from 1912 through 1967 amounted to about 13 feet. One of the obvious beneficial aspects of the District's effort to stop the subsidence was the ever-pressing problem of salt water intrusion into the groundwater basin, especially in the north basin area near Alviso and Palo Alto. The District's current scope of responsibilities has enlarged considerably since those earlier times.

1.6 Present District and Mission

The District has evolved through annexations and mergers with other agencies that were similarly formed to meet the increasing demands placed on the county's water resources. The South Santa Clara Valley Water District was created in 1938 and later renamed the Gavilan Water District. The Central Santa Clara Valley Water Conservation District, formed in 1949, was annexed by SCVWCD in 1954. The Santa Clara County Flood Control and Water District was formed in 1952 to address flood control and water conservation issues. It was governed by the County Board of Supervisors, and became the County Flood Control District in 1964. It merged with SCVWCD in 1968 as the most likely means of achieving closely allied and mutual goals pertaining to the county's water resources issues. The word "conservation" was dropped from the name in 1974, thereby creating the Santa Clara Valley Water District (SCVWD) as it is known today. The Gavilan Water District was annexed by SCVWD in 1987. The District now encompasses 1,300 square miles, operates 10 reservoirs, and serves a population exceeding 1.7 million.

Water conservation using District reservoirs remains one of the paramount strategies of the District to provide an adequate supply of water to its customers. Since the construction of the District's original water conservation facilities in the 1930s, additional facilities have been constructed to improve the capacity, flexibility, and reliability of the system. Responsibilities now embrace proactive stewardship of the greater watershed environment within which any new water supply or flood protection projects, or new environmental programs, are implemented. Areas of increased vulnerability to ecological impacts and water resource degradation are more readily being identified, as are new threats that could easily and adversely impact the quality (and hence usable quantity) of available water supplies.

1.6.1 Increasing Demands

During the period from 1950 to 1965, the county's population roughly tripled, from about 290,000 to nearly 900,000. By 1970 it had exceeded 1 million. In mid-1999, the City of San Jose alone reported a population exceeding 900,000, and the San Jose Mercury News reported it as the sixth-fastest growing large city in the nation. The expansive growth trend began soon after World War II, when the predominantly agricultural economy began transitioning to a combined agricultural-industrial economy. The electronics industry then took root in the 1950s, and it continues its historical growth as the North County's primary economic base. While the agricultural landscape still dominates in the South County and remains viable, it continues to give way to competing interests for more housing and the growing high technology industries.

1.6.2 Imported Water

The first imported water for municipal use delivered to Santa Clara County was in the early 1950s from San Francisco's Hetch Hetchy water system. In the 1960s it served the cities of Palo Alto, Mountain View, Sunnyvale, and Milpitas. Hetch Hetchy's imported water supplied water to communities and agencies in North Santa Clara Valley and also served a precedent for future importation from other sources. The District anticipated continued growth, and with an existing example, looked ahead toward other potential sources.

In 1961 the District signed a contract with the State of California for up to 100,000 acre-feet of water per year from the State Water Project (SWP). As part of the SWP, the South Bay Aqueduct (SBA) was constructed from 1962 to 1965 with its terminus at the base of the foothills of northeastern Santa Clara County, due east of Milpitas. The first SBA water deliveries were made to the county in 1965, and were distributed via the District's Central Pipeline, which terminated at the Vasona Pump Station in Los Gatos, to raw water turnouts and to in-stream and off-stream groundwater recharge sites (percolation ponds) along Los Gatos Creek. However, by the early 1960s, even with the imported water provided from the SWP, it became clear that the District's recharge facilities and groundwater supplies would prove inadequate to meet the ever-increasing demands for potable water imposed by the County's continued growth. The District recognized that it would no longer attempt to expand the recharge ponds; rather, it decided to build one or more treatment plants, as needed, to meet future needs. The District completed construction of the Rinconada Water Treatment Plant in 1967 to treat surface water imported from the SWP. The first phase of the Almaden Valley Pipeline (AVPL) had also been completed by that time, which then conveyed imported water from the Vasona Pump Station to turnouts along the pipeline right-of-way to Guadalupe Creek. The AVPL's subsequent Phase II, completed in 1982, extended to Calero Dam, thereafter providing bi-directional flow. The District was then able to supplement the groundwater supply with treated surface water from an imported source or from Calero Reservoir. The Rinconada plant, in Los Gatos, was designed with a capacity of 80 million gallons per day of potable water.

The SBA was not the only source of water considered for importation. In the early 1950s the proposed San Felipe Division of the federal Central Valley Project (CVP), also known then as the Pacheco Pass project, was considered a hopeful source for future supply. Until the San Felipe Division could be built, however, the SBA and Hetch Hetchy sources were viewed as being capable of resolving immediate problems. In August 1967, the San Felipe Division received final authorization by the U.S. Congress. However, it was not until 1977 that the District signed a contract with the United States for entitlements that would increase over time to a maximum of 152,500 acre-feet per year by 1994. Construction was completed and the first deliveries of San Felipe water to the District were made in 1987. The U.S. Bureau of Reclamation owns and operates the CVP. The District operates and maintains San Felipe Division under contract with the Bureau.

It is interesting to reflect on the economic decisions that were instrumental in driving the District to select its final solution regarding its water shortage. The question that could be asked is, "If the groundwater recharge system that provided such an obvious solution to the original District was so successful, why not simply expand it?" The answer was quite simple, and hinged on overall cost. The ever-increasing demands for potable water, coupled with the limited availability and the incredibly high cost of land for an expanded recharge facility (i.e. significantly increased area), led to the building of the three water treatment facilities that went on line as follows; Rinconada in 1967, Penitencia in 1974, and Santa Teresa in 1989. In retrospect, it was an obvious solution and the correct one for the District to have selected since it was apparent there was a limited amount of land available that was amenable to recharge, plus the high cost of the land acreage that would be required for recharge facilities.

1.7 District System

As the water resources management agency for Santa Clara County, the District manages 10 reservoirs, 19 percolation facilities, three water treatment plants, three pumping stations, a hydroelectric plant, hundreds of miles of rivers, creeks, and other waterways, and an extensive in-county water distribution system of major pipelines that connect sources of imported water, local reservoirs, water treatment facilities, and the treated water pipelines to retailers.

Central to the District's operations are the 10 reservoirs that capture runoff from winter rains, and from which controlled releases provide much of the water to recharge the groundwater basin. Controlled releases from the 10 reservoirs for recharge take place in the drier summer and fall months and are routed to downstream facilities where water can percolate in certain reaches while also serving to maintain ecological balance for aquatic and riparian habitat. Regulated releases from certain reservoirs also provide raw water for treatment at the District's treatment plants. Finally, storage in Anderson and Calero reservoirs serve as the emergency backup to the District's imported water supplies. Table 1-1 shows the 10 reservoirs owned by the District and their significant features, including their watershed, year completed, capacity, surface area when full, and drainage area. They are arranged in decreasing order of capacity. A more complete list of statistics governing our dams, reservoirs, canals and pipelines is presented in Appendix A.

1.8 Construction of Original Dams

The six original dams, including Almaden, Calero, Coyote, Guadalupe, Stevens Creek, and Vasona, were all constructed in 1935/36. Mr. Fred H. Tibbetts, consulting engineer from Campbell, who also became the first chief engineer of the District, was the chief designer of all the dams. The late 1920s and early 1930s was a challenging time for dam design. In the mid-to-late 1920s, many dams were being constructed using hydraulic fill construction techniques, and there had been several catastrophic dam failures. For example, Tibbetts was aware of the 1927 construction failure of the City of San Francisco's Calaveras Dam located behind Milpitas that was being constructed using hydraulic fill techniques. After cleanup of that failure, the hydraulic fill technique was abandoned and the dam construction was completed using tamping techniques by mule and wagon. Tibbetts was also aware of the 1925 failure of Sheffield Dam near Santa Barbara due to liquefaction of the foundation and the catastrophic1928 failure of St. Francis Dam in Los Angeles County, a concrete gravity dam located in southern California. It was this series of dam failures, plus the unprecedented number of dams planned for construction, that led in 1929 to the development of a state agency to specifically oversee the design, construction, and operation of dams in California. Originally, the oversight was under the aegis of the State Engineer; ultimately, an agency was developed within the Department of Water Resources and

Table 1-1 Capacities of District Reservoirs (effective 1998)

Reservoir	Watershed → (tributary to)	Year Completed	Present Capacity (acre-ft) (AF)	Surface Area When Full (Acres)	Drainage Area (Square Miles Above Dam)
Anderson	Coyote Creek → SF Bay	1950	89,073	1,245	193
Coyote	Coyote Creek → SF Bay via Anderson Reservoir	1935	22,925	648	119
Lexington	Los Gatos Creek → Guadalupe River → SF Bay	1952	19,834	475	37
Calero	Calero Creek → Guadalupe River → SF Bay	1935	10,050	347	7
Uvas	Uvas Creek → Pajaro River → Monterey Bay	1957	9,935	286	30
Chesbro	Llagas Creek → Pajaro River → Monterey Bay	1955	8,952	265	19
Stevens Creek	Stevens Creek → SF Bay	1935	3,465	92	17
Guadalupe	Guadalupe Creek → Guadalupe River → SF Bay	1935	3,228	79	6
Almaden	Alamitos Creek → Guadalupe River → SF Bay	1935	1,553	59	12
Vasona	Los Gatos Creek → Guadalupe River →SF Bay	1935	400	57	44

Total Capacity: 169,415 AF

Note: This table does not include the 15 million gallons (46 ac-ft) covered reservoir at the District's Rinconada Water Treatment Plant. Although the dam retaining the reservoir is under DSOD jurisdiction, the reservoir is utilized and operated solely for storage of RWTP treated water production and is not considered as part of the District's potential reservoir capacity. The interested reader is referred to Appendix A for a detailed list of statistics of the District dams and reservoirs.

became known as the State Division of Safety of Dams (DSOD). DSOD had a very active involvement in the design and construction of the District's original dams.

Tibbetts was a brilliant civil engineer who is credited with the vision of developing the six original dams and attendant water recharge facilities surrounding Santa Clara Valley. He was aware of the need to have an engineering geologist directly involved with the design and construction of the dams. He hired Professor C. F. Tolman of the Stanford University Geology Department to perform field inspections both before and during construction. For Coyote Dam, which was knowingly built across the Calaveras fault, Tibbetts hired Chester Marliave to thoroughly review the proposed design. The earthquake-resistant design for Coyote Dam was both innovative and unprecedented, and clearly marked it as the first earthen dam ever designed to knowingly straddle an active fault and to safely withstand tens of feet of potential offset during earthquake shaking. A more detailed description of the innovative earthquake-resistant design aspects of Coyote Dam is presented in Appendix B.

Two more major technical events occurred during the late 1920s and early 1930s that had a profound impact on the design and subsequent performance of District dams. These were the publication of Karl Terzaghi's early articles on soil mechanics (Terzaghi, 1925) and Richard Proctor's 1933 published research (Proctor, 1933) on the compaction of soils that revolutionized the design and construction of earthfills. Ultimately, Tibbetts decided, with the help and insight of Tollman, to build all of the dams of earthfill structures. It is truly impressive to note that even though the original six dams were built 67 years ago, each dam had: 1) detailed field exploration of the dam site and borrow areas prior to construction; 2) detailed laboratory testing performed on the proposed materials for use in construction; and 3) extensive gradation, compaction and permeability testing performed during construction. Detailed geologic maps were developed during construction, especially along the walls of the cutoff trench. In reviewing the design and construction records, it is clear that the original six District dams were probably the first earthfill dams to be built in the United States using modern construction techniques. The construction records for the original dams are replete with voluminous laboratory and field testing, especially focusing on permeability and compaction characteristics of the embankment soils, during both design and construction. A discussion of the construction history for all of the District dams is presented in Appendix B.

Finally, a word about the financing for construction of the dams is in order. By the time Franklin D. Roosevelt was inaugurated on March 4, 1933, the national economy was near rock bottom. Some 15 million jobless and their families (close to one-third of the nation) were suffering hardships and hunger. President Roosevelt's "New Deal" Administration anxiously searched for ways and means to alleviate the problem. One means of combating unemployment and relief was creation of a federal Public Works Administration (PWA) under authority of the National Industrial Recovery Act of 1933. The PWA was administered by the Department of the Interior, and headed by Secretary of the Interior Harold Ickes. The purpose of the PWA and similar organizations created during this period was to increase employment and business activity and to act as "pump priming," thereby aiding the overall economy by stimulating the private sector through the use of public funds.

The PWA made grants to state and local agencies who would hire private contractors to construct the projects. While workers would never become rich, this stop-gap measure did provide for the basic necessities of life. The program and the administration received much criticism, but from the height of the Great Depression in 1933 to 1939, the PWA spent nearly $6 billion on 37,000 projects that provided jobs for an estimated 500,000 persons on a nationwide basis. The Board of Directors of the SCVWCD applied for a PWA grant of $683,000 on November 7, 1932. This grant would constitute a federal contribution of about 30 percent of the total construction cost of the dams and necessary canals for the six reservoirs. In the election of June 19, 1934, Santa Clara County voters approved the first-ever water related bond issue for $2 million for the District. On Junc 30, 1933, official confirmation was received from PWA that the grant for $683,000 had been approved. The final actual construction cost for the original six dams was $1.72 million which, when adjusted for inflation, amounts to an estimated $45.2 million in 2002 dollars. This was a truly remarkable investment in the future of Santa Clara Valley that has paid handsome dividends to those of us who inherited this legacy.

1.9 Report Organization

This report is comprised of an executive summary, four chapters of text, nine appendices, and numerous tables and figures. A brief description of the chapters and appendices following this introductory chapter is presented as follows:

- Chapter 2 presents a brief history of dams and dam regulations in the United States. A chronology of major events involving dams from the period from the early 1600s through the 1996 enactment of the National Dam Safety Act has been prepared.

- Chapter 3 presents a discussion of the dam safety program that is recommended for formal adoption by the District. A discussion is presented of the risks associated with dams, the various elements of a modern dam safety program, inspection guidelines, instrumentation and monitoring guidelines, emergency action plan guidelines, operation plan guidelines, environmental concerns, and measures to reduce the consequences of dam failure.

- Chapter 4 presents the staffing and cost estimates for implementing the Dam Safety Program as presented herein.

- Appendix A presents a summary of the pertinent data for all District dams and reservoirs. Recent aerial photographs for each dam are shown

- Appendix B presents a discussion of the cultural and construction history for all District dams.

- Appendix C presents a discussion of the geologic and seismic issues at District dams. A general discussion of the geologic conditions within the Santa Clara County area impacting our dams is presented. Various types of earthquake shaking are described and pictorially shown. This appendix concludes with some original work that was completed specifically for this report and shows the unique relationship for estimated displacement of District dams during any magnitude of future earthquake. Several different earthquake scenarios are presented and the results can be used for future earthquake planning.

- Appendix D presents a discussion of the District's Earthquake Preparedness and Post-Earthquake Inspection Programs for its dams and water utility system. The appendix includes a copy of the post-earthquake inspection report completed for Anderson Dam following the 2002 Gilroy earthquake, the first such report completed using the new format.

- Appendix E presents a technical discussion of the geotechnical considerations for earth dams and for various types of technical analyses that are commonly used in earth dam engineering. The appendix concludes with recommendations for future static and dynamic analyses for all District dams.

- Appendix F presents a discussion of the history and background of Automated Data Acquisition Systems (ADAS). The appendix concludes with a discussion of what we have learned from the ADAS developed and implemented at Lenihan Dam and presents recommendation for what types of system should be considered for the DIP.

- Appendix G presents a discussion of the types of geotechnical instruments most commonly used for monitoring dam performance. Techniques for instrument installation, operation, and maintenance are presented.

- Appendix H presents the preliminary design of new geotechnical instruments for 10 District dams. These instrumentation designs, which form the basis for the DIP, a major component of the Dam Safety Program, are based on a detailed review of the performance of the dams since construction. Detailed plans and cross sections showing instrument locations are provided for each dam.

- Appendix I presents a summary of all pertinent technical references dealing with dam safety aspects of District dams. The compiled data represents both internal District documents and outside sources. The list was generated based on a computerized search of the District library and central files. The files are presented in chronological order under each dam in alphabetical order.

The main report and appendices are paginated sequentially within each chapter or appendix. Tables are numbered sequentially within each chapter and are presented following their first reference or imbedded directly in the text. Figures are either embedded in the text following their first reference in the text, or presented sequentially at the end of each chapter. Although some of the original figures drafted for this report were of larger size, it was decided to publish the final version in 8.5" x 11" format. References are presented as the last section of each chapter or appendix. The nine appendices are separately identified by consecutive letter designation from A through I and are paginated sequentially with the appendix letter preceding the page number. Tables and Figures in the appendices are similarly identified. A master table of contents for the text and appendices is presented at the beginning of the report, following the frontispiece.

1.10 Performance

This report was written by Richard L. Volpe, Senior Engineer, with the assistance of James L. Nelson, Associate Engineering Geologist, both of whom are with the Dam Safety Section of the Water Utility Engineering Unit. The report was reviewed by several District employees, including Dave Hook, Engineering Unit Manager, Water Utility Engineering Unit; Michael Mooers, Associate Civil Engineer, both from the Water Utility Engineering Unit; Joe Aguilera, Management Analyst, Operations Planning and Analysis Unit; and Kenn Reiller, Senior Engineer, Guadalupe Watershed Program Support Unit. Bob Fields, Emergency Preparedness Coordinator, and Ron Whipp, Risk Management Administrator, reviewed Chapters 1 and 3.

It is intended to invite the California Department of Water Resources, Division of Safety of Dams, for their review and comment prior to publication of the final report.

1.11 References

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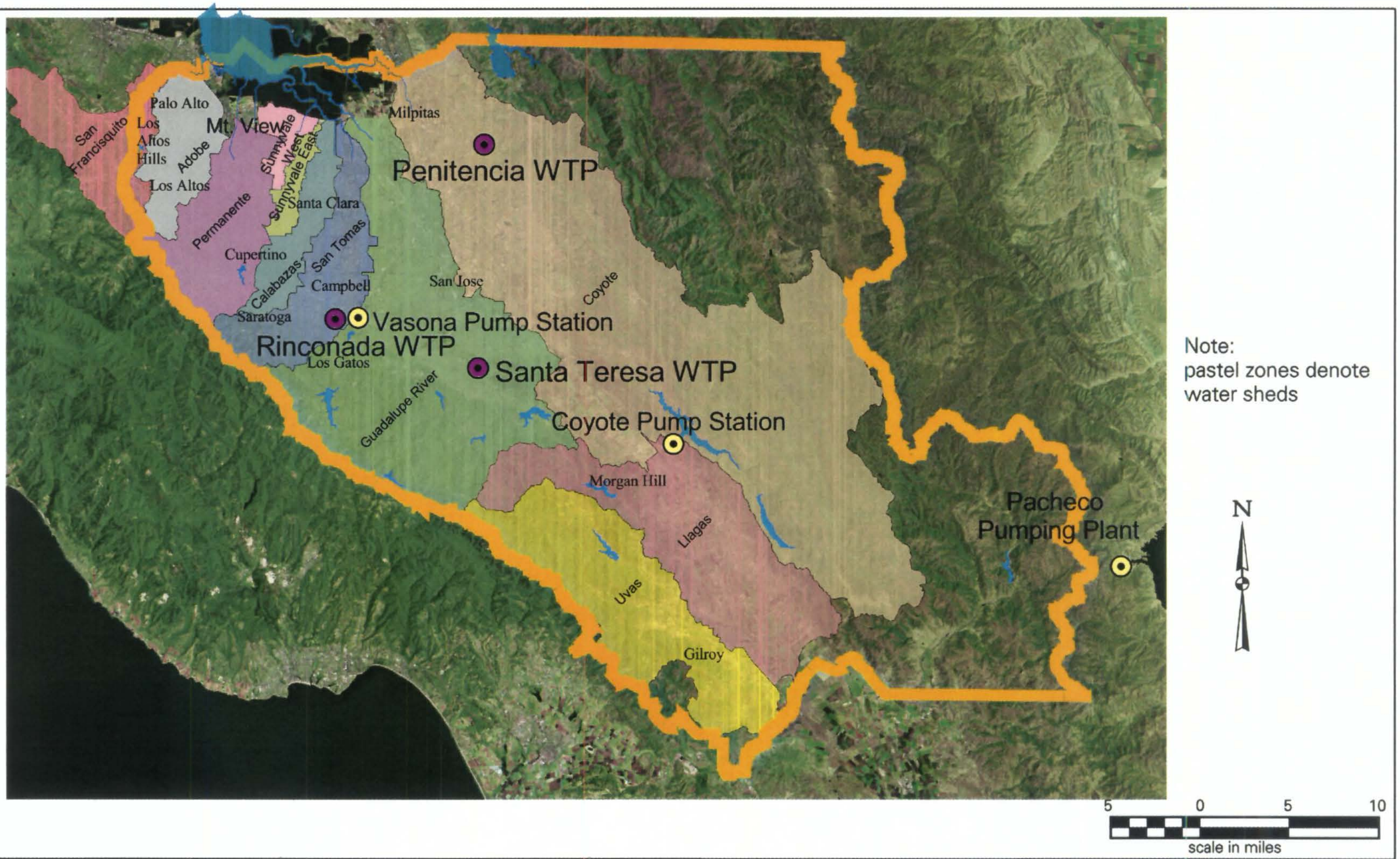

Note:
pastel zones denote water sheds

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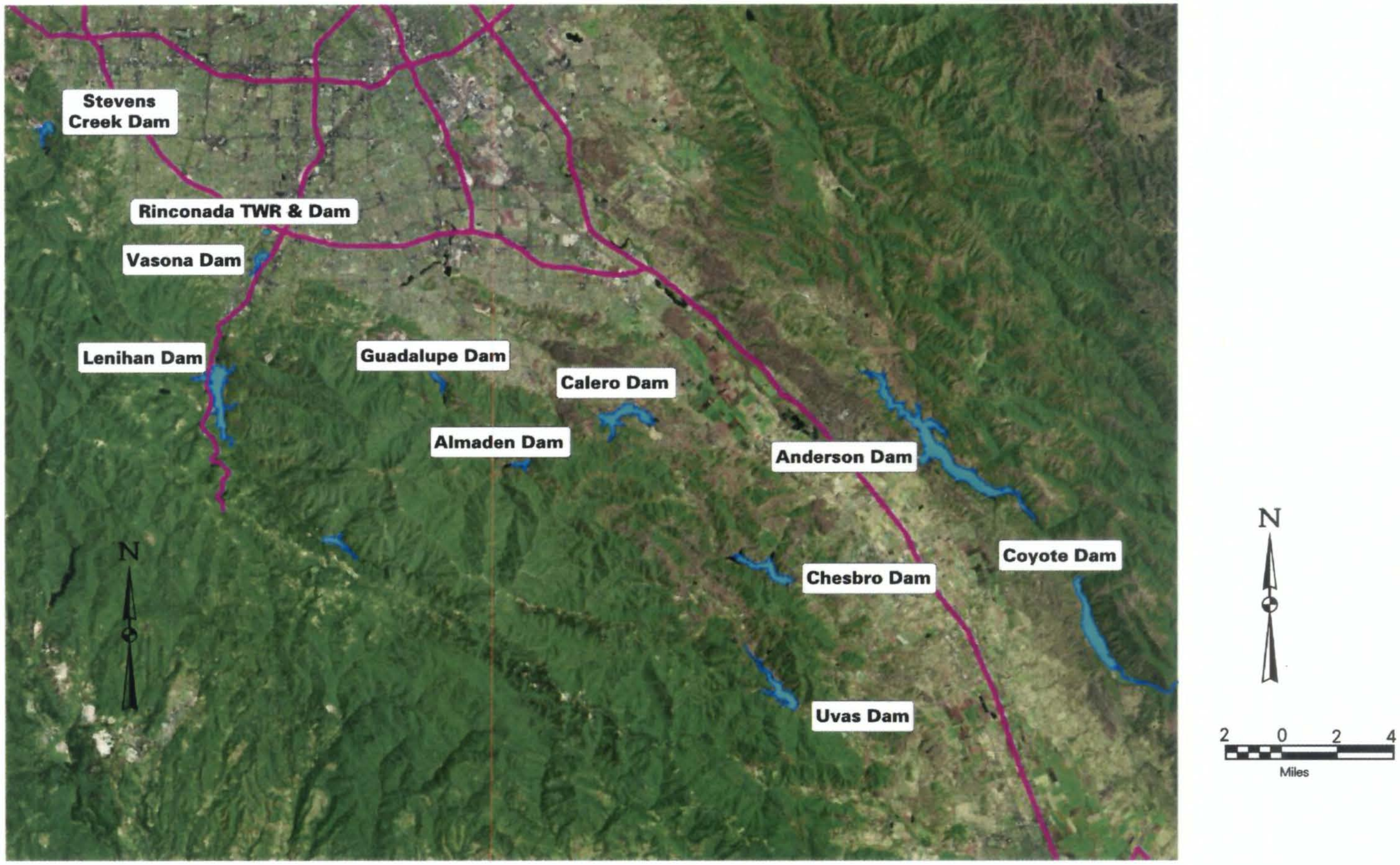
Major District Facilities Map

SCALE as shown

FIGURE 1-1



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Santa Clara Valley Water District

Major District Dams and Reservoirs

SCALE as shown

FIGURE 1-2

Chapter 2

Brief History of Dams and Dam Regulations in the U.S.

2.0 Brief History of Dams and Dam Regulations in the U.S.

2.1 Brief History of Dams

Dams throughout history have been made of timber, rock, earth, masonry, concrete, or combinations of these materials. Timber is seldom used in modern dams because timber is vulnerable to fire and decays with time and timber dams are limited in height. Rock-fill dams consist of an embankment of large rocks which when properly placed are very strong. Since the rock collection is very pervious, rock fill dams must be provided with an impervious element. This is usually accomplished by providing either an impervious clay core or a watertight face on the upstream side made of asphaltic or Portland cement (e.g., concrete-face rockfill dams). Earth dams may be either simple homogeneous embankments or with complex internal zones to safely collect and distribute internal seepage. Masonry and concrete dams are either gravity dams or arch dams (either single-arch or multiple-arch). Gravity dams are dependent upon their own weight for resistance to the pressure of the water from the reservoir. Single-arch dams are curved in the upstream direction to place the concrete in compression and are usually constructed in narrow canyons or gorges where the rocky side walls are strong enough to withstand the tremendous lateral thrust of the dam that is caused by the water pressure. Dams of the multiple-arch type consist of a number of single arches supported by buttresses. Dams may also be constructed with roller-compacted concrete, in which thin layers of concrete are compacted as if they were earth layers; this produces a far stronger dam than a regular earth dam, without the need for full forms as in concrete gravity. The roller compacted concrete is now being proposed for many different sites, from small in-stream overpour sections, to the record setting $200 million, 318 feet high Olivenhain Dam currently under construction by the San Diego County Water Authority.

Dams have been constructed from early times to provide a ready supply of water for irrigation and other purposes. One of the earliest large dams for water supply purposes was a marble structure built c.1660 AD in Rajputana, India. A list of the ten highest dams in the world and the United States is presented on Table 2-1. As shown, the world's current highest dam is the Nurek Dam, which is about 984 feet high and built across the Vakhsh River in eastern Tadjikstan. Rogun Dam, located about 50 miles from Nurek on the same river, is currently under construction and, when completed, will be the first dam ever built to top 1,000 feet in height. The highest dam in the United States is Oroville Dam, which is about 771 feet high across the Feather River in northern California. California is home to five of the ten largest dams in the United States. The largest reservoir in the United States is Lake Mead, the reservoir formed behind Hoover Dam. Fed by the Colorado River, Lake Mead has a capacity of 28.5 million acre-feet of storage. The world's largest reservoir will be formed behind the Three Gorges Dam, which is currently under construction on the Yangtze River in China. Started in 1994, the project is expected to take 20 years and cost $24.7 billion. The power generation is expected to be an incredible 18.2 million kilowatts. When completed, the 590 feet high dam will produce a reservoir that will inundate about 437.5 square miles and create a reservoir about 350 miles long.

Table 2.1 - Highest Dams in the World and the United States

A) World's Highest Dams

Rank	Name of Dam	River	Country	Type	Height (m/ft)	Year Completed
1	Rogun	Vakhsh	Tadjikistan	E-R	335/1099	Under Const.
2	Nurek	Vakhsh	Tadjikistan	E	300/984	1980
3	Grande Dixence	Dixence	Switzerland	G	285/935	1961
4	Inguri	Inguri	Georgia	A	272/892	1980
5	Vajont	Vajont	Italy	A	262/860	1960
6	Chicoasen	Grijalva	Mexico	E	261/856	1980
7	Tehri	Bhagirathi	India	E	261/856	Under Const.
8	El Gallinero	Tenasco	Mexico	G	260/853	1946
9	Mauvoisin	Drance de Bagnes	Switzerland	A	250/820	1957
10	Alberto Lleras C.	Guavio	Colombia	R	243/797	1989

B) U.S. Highest Dams

Rank	Name of Dam	River	State	Type	Height (m/ft)	Year Completed
1	Oroville	Feather	California	E	235/771	1968
2	Cyprus	Bruno Creek	Idaho	E	226/741	1982
3	Hoover	Colorado	Nevada	A-G	223/732	1936
4	Dworshak	North Fork Clearwater	Idaho	G	219/718	1973
5	Glen Canyon	Colorado	Arizona	A-G	216/709	1964
6	New Bullards Bar	North Yuba	California	A	194/636	1969
7	New Melones	Stanislaus	California	E-R	191/627	1979
8	Shasta	Sacramento	California	E-R-G	183/600	1945
9	New Don Pedro	Tuolumne	California	R	178/584	1971
10	Yankee Doodle Tailings	Yankee Doodle & Silver Bow	Montana	R	174/571	1972

Type:
E = Earthfill
R = Rockfill
E-R = Earth and Rockfill
E-R-G = Earth and Rockfill Gravity
G = Concrete Gravity
A = Concrete Arch
under const. = under construction
1 m = 3.28 feet

Most modern dams are constructed for multiple purposes. These may include providing storage capacity for irrigation, flood control, recreation, improvement of the navigability of waterways, and furnishing power for hydroelectric plants. Notable dams built to provide hydroelectric power include the Aswan Dam in Egypt, the Kariba Dam in Zambezi, the Daniel Johnson Dam in Canada, the Guri Dam in Venezuela, and the Itaipú Dam between Brazil and Paraguay, which at 623 ft (190 m) produces more than 12,600,000 kilowatt (kW) of electricity, is the currently the largest hydropower dam in the world. The Grand Coulee Dam, located near Spokane, Wash., is the largest hydropower dam in the United States with a production capacity of 6,465,000 kW of electricity. Grand Coulee Dam was built just prior to WWII and is thought by Reisner (1986) to be the primary reason the U.S. was able to out produce war materials and thus defeat Japan and Germany. The 20th century witnessed many great dam projects in the United States, including the Central Valley project in California, the largest water conveyance project in the world; the Missouri River Basin Project; and the Tennessee Valley Authority. Another notable water project in the 20th century includes the construction of a large dam in Panama which forms Gatún Lake, the key to the Panama Canal system. Based on the October 2001 update of the National Inventory of Dams, the United States has 74,262 dams.

2.2 Dam Safety Issues in the United States

While dams provide tremendous benefits to the local and national infrastructure, they also represent one of the greatest risks to public safety, local and regional economies, and the environment. Historically, some of the largest disasters in the U.S. have resulted from dam failures. In 1889, 2,209 lives were lost when the South Fork Dam failed above Johnstown, Pennsylvania. The 1928 St. Francis Dam failure in southern California killed more than 500. During the 1970's the Buffalo Creek, Teton, and Kelly Barnes dam failures collectively caused the deaths of over 175 persons and over $1 billion in property damage. A list of the historically significant history incidents that have shaped dam safety in the United States is presented on Table 2-2.

In order to provide safe continuing service, dams require ongoing maintenance, monitoring, frequent safety inspections, and rehabilitation. Over 90% of the nation's approximately 75,000 dams are regulated by the states. In order to protect public safety, state dam safety programs must be adequately staffed and funded. Alabama and Delaware are the only two states without dam safety programs. Many others lack the human resources to adequately enforce regulations. As discussed in more detail later in this chapter, this is not the case in California, which has perhaps the most pro-active, well-staffed, and expert dam safety organization's in the world. It is unfortunately true, however, that many dam owners, including most of the private dam owners who own over half of the dams in the United States, lack the financial resources necessary to perform dam maintenance or to make significant repairs.

Like all structures built by man, dams deteriorate with time. Inadequate attention to maintenance causes dams to become more susceptible to failure. In the past three years, since January 2000, more than 520 dam safety incidents, including 61 dam failures, were reported to the National Performance of Dams Program. This is a Federal Emergency

Table 2-2 Major Events in Dam Safety in the U.S.

DATES	EVENTS
1600	Spanish colonists made diversions in New Mexico on the Rio Grande River and other streams.
1634	One of the first American water power projects was completed at Lower Falls, Neponset River, Massachusetts.
1824	U.S. Army Corps of Engineers began water project development on the Ohio and Mississippi Rivers.
1868	Compaction in earthfill construction first recommended.
1874	Failure of the Mill River Dam in Massachusetts due to uncontrolled seepage.
May 31, 1889	Failure of South Fork Dam in the Allegheny Mountains in Pennsylvania due to overtopping; resulting Johnstown Flood killed 2,209.
1913	W. E. Fuller proposes a flood frequency formula for flood estimation.
Jan. 27, 1916	Failure of Lower Otay Dam in San Diego, California due to flood overtopping.
June 29, 1925	Failure of Sheffield Dam in Santa Barbara, California due to seismic forces.
Mar. 12, 1928	Failure of St. Francis Dam in Los Angeles County, California focuses public scrutiny on safety of dams in the U.S. The failure and resulting flood led to the deaths of at least 500 persons. The Governor of California authorizes the Division of Safety of Dams.
1933	The Proctor test to evaluate compaction of soils revolutionized the design and construction of earthfills.
Dec. 14, 1963	Failure of Baldwin Hills Reservoir Dam in southern California. First dam failure to be presented "live" on television.
1965	The Federal Power Commission issued Order No. 315 requiring owners to ensure safe construction and operation of dams.
Feb. 26, 1972	Failure of a coal refuse dam at Buffalo Creek, West Virginia led to deaths of about 125 persons.
Aug. 1972	Public attention to the hazards created by water reservoirs led to the enactment of the National Dam Inspection Act (Public Law 92-367).
May 1975	The Army Corps of Engineers issued a report, "National Program of Inspection of Dams" including an inventory of U.S. dams and inspection guidelines, inspection, evaluation, and modification.
Jun. 5, 1976	Failure of Teton Dam in Idaho due to internal erosion. The resulting failure and flood led to the deaths of 11 persons. The failure of this dam, designed and constructed by the U.S. Bureau of Reclamation, led to widespread review by federal agencies regarding dam safety.
Nov. 6, 1977	Failure of the Kelly Barnes Lake Dam near Toccoa, Georgia. There were 39 fatalities due to the resulting flood.
1979	Federal Guidelines for Dam Safety were published.
1982	U.S. Committee on Large Dams (USCOLD) passed a resolution urging state governments to give high priority to enacting dam safety legislation and to allocating resources to dam supervision.
April 24, 1984	Morgan Hill earthquake near San Jose, California brought new attention to dam safety due to the acceptable performance of Coyote and Anderson Dams, which safely withstood the highest accelerations ever recorded in embankment dams.
1985	A National Research Council report titled "Safety of Dams: Flood and Earthquake Criteria" was published.
1985	The Association of State Dam Safety Officials (ASDSO) became active. The Interagency Committee on Dam Safety (ICODS) was formed.
October 17, 1989	Loma Prieta earthquake near Santa Cruz, California brought new concerns to dam safety performance during earthquakes. Austrian Dam and Lexington Dam, along with 6 other dams located near earthquake epicenter, suffered damage.
1994	National Performance of Dams Program (NPDP) officially started.
Oct. 12, 1996	The Water Resources Development Act of 1996 was signed into law and established a National Dam Safety Program (Section 215 of Public Law 104-303).
Feb. 28, 2003	Publication of Dam Safety Program report by Santa Clara Valley Water District. First known report of its kind in U.S. by a dam owner.

Management Agency (FEMA) sponsored agency that collects and archives information on dam performance as reported by state and federal regulatory agencies and dam owners. Dam safety incidents are events such as large floods, earthquakes or inspections that alert dam safety engineers to deficiencies that threaten the safety of a dam. Due to limited state staff, many incidents are not reported, and therefore the actual number of incidents is likely to be much greater.

The National Dam Safety Program Act of 2000 defines four possible ratings for the potential hazard of dams, as summarized on Table 2-3. As seen by these definitions, the Class 1 or High Hazard Dam is defined as any dam whose failure would cause a possible loss of human life. The number of dams with a high-hazard rating potential is increasing in the United States. In 1998, states reported 9,281 high-hazard potential dams, with North Carolina having the highest number (874). California reported a total of 1512 dams, of which 1220 were regulated by DSOD and 272 were given a Class 1 or High Hazard Rating. As downstream land development increases, so will the number of high-hazard potential dams. As these dams often require major repair to accommodate more stringent inspection, maintenance and design standards, financial support for state dam safety programs will have to keep pace. Even more alarming, states presently report approximately 2,100 "unsafe" dams, which have deficiencies that leave them highly susceptible to failure. The number of unsafe dams in North Carolina has increased from 40 in 1998 to 139 in 2001. Many other states have large numbers of unsafe dams including Ohio (450), Texas (403), and Colorado (190). California did not have any dams designated as unsafe. Eleven state agencies do not keep statistics on unsafe dams; therefore the actual number is potentially much higher.

Table 2-3 Dam Hazard Ratings

Rating	Description	Hazard Definition
Class 1	High	Those dams located where failure may cause loss of human life or major damage to dwellings, commercial or industrial buildings, main railroads, important public utilities, or where a high risk highway may be affected or damaged. This classification must be used if failure may result in the loss of human life.
Class 2	Significant	Those dams located where failure may cause minor damage to dwellings, commercial or industrial buildings, important public utilities, main railroads, or cause major damage to unoccupied buildings, or where a low risk highway may be affected or damaged. The potential for loss of human life resulting from failure of a Class 2 dam must be unlikely.
Class 3	Low	Those dams located in rural or agricultural areas where failure may cause minor damage to nonresidential and normally unoccupied buildings, or rural or agricultural land.
Class 4	Negligible	Those dams where failure is expected to have no potential for loss of human life, no potential for property damage and no potential for significant harm to the environment.

The combined effect of rapid downstream development and aging/non-compliant structures demands fully funded and staffed state dam safety programs as well as substantial and pro-active funding for dam repairs. In October 2001, the American Society of Civil Engineers (ASCE) issued a "Report Card" on infrastructure within the United States. The following are highlights from that document specifically related to dams.

- **Funding for Dam Rehabilitation** - Since the last ASCE Report Card, Congress has passed the Small Watershed Rehabilitation Act of 2000, which authorizes $90 million over five years to assess and rehabilitate a portion of the 10,000 dams constructed by the USDA. While this law is a positive step, it is neither a comprehensive nor a permanent solution. According to preliminary results of a study by the Association of State Dam Safety Officials (ASDSO), the total investment to bring U.S. dams into safety compliance or remove those that are no longer needed tops $30 billion. Except for a handful of state programs offering low interest loans to dam owners, there are no external funding sources for dam rehabilitation or repair. Private owners have the greatest need of funding. The Small Watershed Rehabilitation Act addresses less than 10% of our nation's dams; the remaining 90% demand similar attention.
- **Implementation of the National Dam Safety Program** - Since the last ASCE Report Card, the National Dam Safety Act of 1996 has been implemented. Under this program, state dam safety agencies have received grants totaling $7 million to assist them with improving dam safety regulatory programs by procuring equipment, implementing new technology, and enabling more frequent inspections. The program also provided opportunities for continuing education to dam safety engineers and funding for research to advance the technology of investigations, construction and rehabilitation of dams.
- **Problem Areas** – Prior to 1998 few state dam safety programs were adequately funded or staffed. Currently (2001) that situation remains essentially the same. On average nationwide, there are 582 state-regulated dams per single full-time equivalent (FTE) staff. In nine states, this number exceeds 800, and six of these report over 1,000 dams per FTE staff. In Texas, there are still only six staff members responsible for inspecting over 7200 dams. Two states, Alabama and Delaware, still have not passed dam safety legislation. As a result, an estimated 2,000 dam structures, perhaps more, are unregulated. At last count, approximately 200 of these structures were classified as high-hazard. The Association of State Dam Safety Officials estimates that over 10,000 dams avoid regulation because of special state exemptions (none in California), such as agricultural uses, regardless of size or hazard potential. These dams are not inspected and their condition and hazard potential are unknown. California, due to its strictly enforced laws and oversight due to high earthquake potential, leads the nation with a total of 65 FTE staff, or 19 dams per FTE.

The potential for damage due to dam failures is increasing with the increase of residential and commercial development downstream of dams and the aging of the dams. In many cases, existing dams will need to be modified to keep downstream areas safe from disaster. Table 2-4 presents a list of the more notable dam failures that have occurred in the United States within about the past 30 years.

Table 2-4 Recent Notable U.S. Dam Failures

Year	Name of Dam	Location	Deaths	Damage
1972	Canyon Lakes	South Dakota	278	$60 million
1972	Buffalo Creek	West Virginia	125	$400 million
1976	Big Thompson Creek	Colorado	139	$60 million
1976	Teton	Idaho	11	$400 million
1977	Kelly Barnes	Georgia	39	$30 million
1978	Texas Hill County	Texas	25	$25 million
1982	Lawn Lake	Colorado	3	$21 million
1990	Shadyside	Ohio	24	$8 million

There were about 200 notable dam and reservoir failures in the world in the 20th century. More than 8,000 people died in these disasters. As the public becomes more aware of the importance of dam safety, increasingly more emphasis is being placed on emergency action plans. Since failure of a dam can take only hours or sometimes minutes, it is imperative to have a detailed plan of action ready for use in the event there is a threat to the dam. Dam owners and local officials must be prepared to act promptly and effectively when a dam begins to show signs of failure. Early identification of a hazardous situation may provide valuable additional time to warn and evacuate downstream residents and to implement measures to prevent or delay a dam failure.

2.3 History of Dam Failures in California

As shown on Table 2-2, there were several dam failures that occurred early in the 20th century in southern California that had a profound affect on dam safety issues, especially in California. The first of these failures was the Lower Otay Dam, located near San Diego, which failed in 1916 due to flood overtopping (inadequate spillway size). This was quickly followed by the 1925 failure of Sheffield Dam near Santa Barbara, the only confirmed earth dam to have failed due to earthquake forces. St. Francis Dam was built by the City of Los Angeles Bureau of Water Works and Supply (precursor to LADWP) in 1925-26 as a curved concrete gravity dam, approximately 200 feet high in San Francisquito Canyon, about 5 miles northeast of what is now Magic Mountain, California. The stated purpose of the dam was to provide an additional 38,000 acre-feet of storage for the Los Angeles - Owens River Aqueduct water in close proximity to Los Angeles. The dam failed catastrophically upon its first full filling, near midnight on March 12/13, 1928, killing at least 450 people in the San Francisquito and Santa Clara River valleys. It was the greatest American civil engineering failure in the twentieth century. The failure of St. Francis Dam was one of a number of important dam failures that occurred in the 1920s and 30s, when American civil engineers began to push the limits of dam construction technology then in its infancy. The dam's high-profile failure led to the immediate and irrevocable decline of William Mulholland, architect of the Los Angeles water supply system. Like most notorious engineering failures, looking back we can take some measure of satisfaction in knowing that considerable long-term societal benefit resulted from public outcry following the disaster. Some of the most important consequences included: a) the formulation of the world's first dam safety agency, the California Division of Safety of Dams; b) normalization of uniform engineering criteria for testing of compacted earthen materials that is still in use today on a world-wide basis; c) a reassessment of all Los Angeles Department of Water and Power dams and reservoirs which led to an extensive retrofit of Mulholland Dam; and d) the formulation of a state-mandated process for arbitration of wrongful death suits that formed the basis of similar legislation following the 1989 Loma Prieta earthquake.

2.4 Formation and Jurisdiction of DSOD

As a direct result of the 1928 failure of St. Francis Dam in Southern California, and because of the exposure to potential risk to the general populace from a number of water storage dams in California, the Legislature in 1929 enacted legislation providing for supervision over non-federal dams in the State. Before the enactment of this legislation, State supervision over dams was exercised by either the State Engineer or the State Railroad Commission. This supervision was limited in scope and extended to less than half of the dams in the State. The statute enacted in 1929 provided for (1) examination and approval or repair of dams completed prior to the effective date of the statute, August 14, 1929; (2) approval of plans and specifications, and supervision of construction of new dams, and of the enlargement, alteration, repair, or removal of existing dams; and (3) supervision over maintenance and operation of all dams of jurisdictional size.

Dams under jurisdiction of the State include any natural or artificial barrier, together with appurtenant works, which are 25 feet or more in height or have an impounding capacity of 50 acre-feet or more. Any artificial barrier not in excess of 6 feet in height, regardless of storage capacity, or that has a storage capacity not in excess of 15 acre-feet, regardless of height, is not considered jurisdictional. The relationship of reservoir size and height of dam constituting the jurisdictional size relationships is presented in Figure 2-1. In 1963, the failure of the Baldwin Hills Dam, in Los Angeles, caused the Legislature to amend the Water Code to place both new and existing off-stream storage facilities under State jurisdiction.

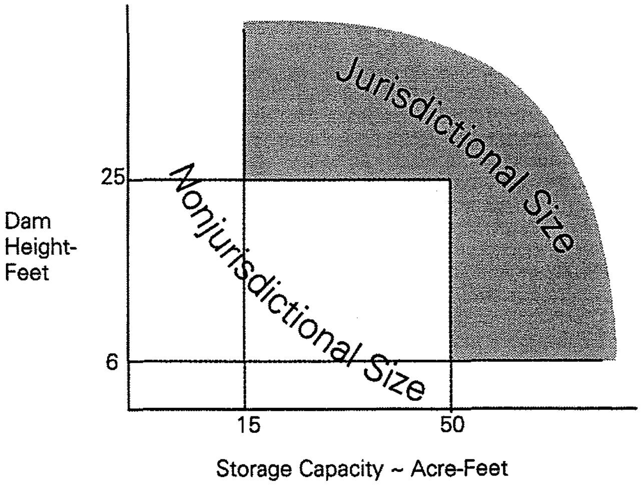


Figure 2-1 - DSOD Jurisdictional dams

The statutes governing dam safety in California, Division 3 of the Water Code places the supervision of the safety of non-federal dams and reservoirs under the jurisdiction of the Department of Water Resources' Division of Safety of Dams (DSOD). In administering Division 3 of the Water Code, DSOD must comply with provisions of the California Environmental Quality Act of 1970. In compliance with that Act, all formal approval and revocation actions must be preceded by appropriate environmental documentation.

DSOD requires that all plans and specifications for the construction of new dams, or for the enlargement, alteration, repair, or removal of existing dams, must be reviewed by them. Before the owner can proceed with construction, enlargement, alteration, repair, or removal, written authorization must be obtained. The Department of Water Resources must issue a certificate of approval based upon the findings of its personnel before water can be impounded behind a new dam or behind an existing dam which has been enlarged, altered, or repaired. These certificates may contain restrictive conditions and may be amended or revoked by the Department of Water Resources at any time for non-compliance. Operating dams are currently inspected by DSOD personnel twice per year to assure that they are adequately maintained and to direct the owner to correct any deficiencies found. These inspections are discussed in more detail in Chapter 3 of this report. As part of the license agreement for each dam, DSOD requires that dam owners, on an annual basis, conduct a comprehensive safety review and submit a detailed written safety report to DSOD. The safety report must document and provide interpretation of all current and historical instrumentation readings, any changes in reservoir operation, changes or modification to the dam or appurtenant structures, and any proposed changes to future instrumentation or reservoir operation.

California has a very pro-active dam safety agency in DSOD. They have continued to work with dam owners in the state to promote dam safety and, after meeting with them to discuss this Dam Safety Program, encouraged the district to complete the effort and the published report. DSOD also completed a cursory review of the DIP summarized in Appendix G and encouraged the district to proceed with the installation of new instrumentation. DSOD concurs that the implementation of a fully automated instrumentation system at all district dams will significantly increase and improve our understanding of our 11 dams.

2.5 Federal Jurisdiction of Dams

2.5.1 National Dam Safety Program Act

On October 12, 1996, the Water Resources Development Act of 1996 (Public Law 104-303) was enacted into law. Section 215 of this law established a National Dam Safety Program and named the Director of FEMA as its coordinator. The purpose of the National Dam Safety Program, as expressed in Section 215(a) of Public Law 104-303, is to "reduce the risks to life and property from dam failure in the United States through the establishment and maintenance of an effective national dam safety program to bring together the expertise and resources of the federal and non-federal communities in achieving national dam safety hazard reduction." The objectives of the National Dam

Safety Program are to:

- ensure that new and existing dams are safe through the development of technologically and economically feasible programs and procedures for national dam safety hazard reduction;
- encourage acceptable engineering policies and procedures to be used for dam site investigation, design, construction, operation and maintenance, and emergency preparedness;
- encourage the establishment and implementation of effective dam safety programs in each state based on state standards;
- develop and encourage public awareness projects to increase public acceptance and support of state dam safety programs;
- develop technical assistance materials for federal and non-federal dam safety programs; and,
- develop mechanisms with which to provide federal technical assistance for dam safety to the non-federal sector.

2.5.2 Federal Energy Regulatory Commission

The Federal Energy Regulatory Commission is an independent regulatory agency within the Department of Energy that performs the following functions:

- Regulates the transmission and sale of natural gas for resale in interstate commerce;
- Regulates the transmission of oil by pipeline in interstate commerce;
- Regulates the transmission and wholesale sales of electricity in interstate commerce;
- Licenses and inspects private, municipal and state hydroelectric projects;
- Oversees environmental matters related to natural gas, oil, electricity and hydroelectric projects;
- Administers accounting and financial reporting regulations and conduct of jurisdictional companies, and;
- Approves site choices as well as abandonment of interstate pipeline facilities.

FERC was created through the Department of Energy Organization Act on October 1, 1977. At that time, the Commission's predecessor, the Federal Power Commission (FPC), was abolished, and the new agency (FERC) inherited most of the FPC's responsibilities.

The Commission's legal authority comes from the Federal Power Act of 1935, the Natural Gas Act (NGA) of 1938, the Natural Gas Policy Act (NGPA) of 1978, the Public Utility Regulatory Policies Act of 1978, and the Energy Policy Act of 1992.

FERC has jurisdictional authority over just one of the district dams; namely Anderson Dam. FERC assumed jurisdictional authority over Anderson Dam in 1987 when the district completed construction (the preliminary permit had been granted in 1982) of a small powerhouse located about ½ mile from the downstream toe and adjacent to Coyote Creek. In a manner similar to DSOD, the FERC also requires each dam owner to perform an annual safety inspection and submit an annual surveillance report. In addition, FERC also requires the district to submit a detailed safety review report prepared by an independent consultant every five years. Previous five year reports were completed in 1986, 1991, 1996 and 2001. The five year report must be completed by an independent consulting engineer who is responsible to review and document all design and safety-related issues that have occurred since the initial granting of the operational license (in our case 1987) and affirm that the dam is in compliance with the then current FERC dam safety requirements. For example, in 2002 the FERC advised the district that it had a new method of assessing potential flood inundation maps in the event of a dam failure and requested that the district perform such analyses, prepare new maps, and resubmit the Emergency Action Plan (EAP). The district has completed the required analyses and is currently (Feb. 2003) in the process of updating the EAP and inundation maps and will submit a final report by early this summer.

2.6 References

ASCE Report Card on Dam Safety Issues in the United States
http://www.asce.org/reportcard/index.cfm?reaction=factsheet&page=8

National Dam Safety Program Act (P.L. 106–580, Dec. 29, 2000)
http://epw.senate.gov/dam.pdf

Chapter 3

Dam Safety Program

3.0 DAM SAFETY PROGRAM

3.1 Introduction

This chapter presents a discussion of the various elements that constitute the dam safety program recommended for adoption by the District and how these elements will be implemented and/or augmented in the current program. The District has a 67-year long proud history of successfully operating and maintaining its dams. During this period, the dams have been maintained in virtually continuous operation serving the water needs of Santa Clara County. During that period, there has not been one recorded incident of threatened dam safety or unanticipated release of water from any of the District dams. In 1998, an internal situational emergency was declared at Lenihan Dam in order to mobilize management resources in response to unanticipated high pore pressures that were discovered acting adjacent to the outlet pipe. However, this response and mobilization might not have been warranted if the dam had been instrumented as it is today. Much of the credit for these past District accomplishments goes to employees who have since retired, and to previous and current members of our Board of Directors who value and respect this vital District activity. Their legacy of excellence is certainly recognized and will be memorialized with the formal adoption of this Dam Safety Program.

There is no doubt that future improvements will be made in the area of more sophisticated engineering analyses, and in the development of new instruments and recording devices, to help the next generation of dam safety staff to better measure, monitor, and analyze dam performance. Nevertheless, it is our responsibility today, using current tools and methodologies, to set a course of action that guarantees success of the program. The first step of this journey is the completion and publication of this report. The adoption of this report and its basic recommendations for new and modern instrumentation for all District dams, along with the development of a well-trained staff to implement, operate and maintain the Dam Safety Program, will provide assurance that all District dams are being operated in the safest possible manner to ensure public safety for all Santa Clara County residents.

3.2 Objectives of the Program

The objectives and commitments of the Dam Safety Program include the following commitments and related work elements:

- All dams and appurtenant structures will be operated in a reasonable manner so as to keep them operationally and structurally safe;
- All dams will be maintained to safe and non-defective condition to prevent degradation of the dam or appurtenant structures or unnecessary harm to the environment;
- All dams will have a planned maintenance program jointly developed by watershed maintenance, dam safety, vegetative maintenance, and utility maintenance staff;

- Individual dam maintenance records will be managed and maintained by watershed management staff of the Watershed Management Division;
- All District dams will be inspected at least twice per year by watershed management and Dam Safety Section personnel with written reports prepared for the dam owner that summarize and document the results of the inspections;
- All dams and their foundations will be well instrumented with electric piezometers, automated inclinometers, and data acquisition systems to enable the determination that internal pore pressures and deformations remain within safe bounds, and that these instruments can be monitored during future earthquakes;
- All dams will have a series of crest monuments surveyed on a regular basis to ensure that the dam is not moving;
- Surveillance reports summarizing each dams reservoir fluctuations, instrumentation response, and any alarm responses, will be prepared for the dam owner and DSOD/FERC at least once per year, or more frequently as dictated by circumstances;
- Emergency Action Plans will be prepared for each District dam, and revised and/or updated as necessary;
- Staff volunteers to the Earthquake Preparedness Program will be trained in effective ways to monitor and inspect District dams following a major earthquake or other significant impact to any of our dams. Unique earthquake preparedness kits will be prepared for each dam that focus on the providing District staff with all necessary tools to efficiently respond;
- All dams will have their static and dynamic stability evaluated and reviewed once per 5-10 years, or more often as appropriate;
- Alls dams will have their Probable Maximum Precipitation (PMP) and Probable Maximum Flood (PMF) scenarios evaluated and reviewed once per 5-10 years, or more often as appropriate;
- It would be desirable to assess the feasibility of installing permanent (24/7) Global Positioning Satellite (GPS) stations located at Coyote, Anderson, and Lenihan Dams to ensure that these facilities are not being severely impacted by tectonic (fault related) movements;
- All dams will have a Failure Mode Analysis performed once per 5-10 years, or more often as appropriate;
- The District will maintain a well-staffed Dam Safety Section, under the direction of the Water Utility Operations Division, whose responsibility it will be to monitor the instrumentation and automation system, assist in the development and monitoring of the maintenance program, assist in all dam safety inspections, perform as the principal District contact for all dam safety-related issues, take the lead in situation intelligence activities for District dam emergencies, complete the training and other related functions associated with the Earthquake Preparedness Program, and to prepare timely and detailed surveillance reports documenting safety-related issues for all District dams.

The practice of civil and earth dam engineering has changed dramatically in the 65+ years since the original six dams were built at the District. With the advent of modern computers and the development of specialized software programs, we can now conduct detailed engineering analyses very quickly and perform detailed analyses, such as dynamic finite element analyses, that were not even considered back then. The use of computers has also revolutionized the area of geotechnical instrumentation. As further discussed in Appendix G, Preliminary Instrumentation Design, we propose to install over 150 new piezometers for measuring pore pressure, along with inclinometers to measure movement, and accelerometers to measure earthquake accelerations, at ten District dams over the next decade. These new instruments will augment the over 40 piezometers already installed at Lenihan Dam. These instruments will replace the limited number of older instruments we currently have installed in our dams. Such an aggressive and comprehensive instrumentation program as the DIP would not even be considered feasible or affordable without the ability to have the entire data collection program automated by computer.

Currently, we have a dedicated staff who manually collect, reduce, compute, record, store, and transmit the data from the existing instruments on a quarterly basis, except from Lenihan Dam. When the DIP is complete, we will be receiving geotechnical instrumentation data from over 200 instruments at 11 dams collected on 6-hour increments. As we have already discovered from the automated instrumentation results collected at Lenihan Dam during the past three years, a careful review of the instrumentation data during reservoir filling and retreating is analogous to us measuring and recording the heartbeat of the dam, all done automatically. The point is that some aspects of dam safety, such as instrumentation, are new and very exciting. However, dam safety is not just about instrumentation or high speed computers. Other aspects of the program, such as safety inspections, routine maintenance, planned maintenance, and safe reservoir operation, have not changed in concept since our dams were originally built. The overall concept of a modern dam safety program is to have a collection of written standard operating procedures and methodologies that will be followed by all those involved with any aspect of dam safety at the District, and to produce written reports on an annual or more frequent basis, to both senior management and regulators, that completely documents the safe performance of each dam.

As mentioned in Chapter 2, there are about 75,000 dams in the United States, ranging in height from tens to nearly 800 feet high, and with reservoirs capacities ranging from less than ten to greater than 2 million acre-ft. These statistics indicate that there is a continuing need for dam safety in the United States. Most dams are considered to represent essential elements of our national infrastructure, but the risk to public safety in case of failure is great, and it is real. Large and growing numbers of lives and valuable properties are at stake. Consider for example the growth of Santa Clara County since the District's initial six dams were built 67 years ago, and the incredible growth and development of Silicon Valley over just the past several decades. The population of San Jose in 1936 was about 65,000 when our first six dams were built. In the most recent census in 2000, the city's population is reported to be about 895,000, or nearly a 14-fold increase. Dramatic changes have occurred in land use. Today, businesses with a net worth of tens of billions of dollars are located downstream of our dams where once there were vast stretches of orchards. Furthermore, the general public has little or no knowledge of the significance or what it means to have major dams located upstream of them. Although there are many who are concerned about dam safety, the principal responsibility of safe dam operation and maintenance usually rests with the dam owner.

Dam owners, like the Santa Clara Valley Water District, serve society by meeting important local needs, such as water treatment, water supply and conservation. In the process of operating and selling water from the reservoirs, the District produces revenue and takes the excess revenues, less cost, and invests it in other essential activities as a steward of the watersheds it manages. However, these reasons do not and can not justify the benefits of dam ownership if the owner does not provide safety for people and property downstream of these structures. The cost associated with dam safety is very small in comparison to the potential damages that would follow dam failure. The situation is even more potentially costly here in the Santa Clara Valley which has such an expensive infrastructure, and where there is a potential for high loss of life and extensive property damage.

The District's Board of Directors has historically recognized the tremendous value of our dams and reservoirs and just recently provided in its Governance Policies that "... *The mission of the District is a healthy, safe, and enhanced quality of living in Santa Clara County through the comprehensive management of water resources in a practical, cost-effective, and environmentally-sensitive manner*." Individual dam owners like the District have a direct and positive influence on the safety of a dam depending on how they are operated, maintained and monitored. Owners can and should develop their own dam safety program, which should include such important elements as routine inspection, monitoring through instrumentation, proper operation and maintenance of the structure, and the appropriate emergency action planning and preparation. Such a program is directly related to the type of dam structure and its immediate environment and depends on the owner's knowledge of the dam, how it works, the critical issues impacting overall safety, and the downstream risks.

3.3 Hazards, Risks, and Failures

Present national loss statistics from dam failure fully justify the need for dam owners to better understand the risks involved with dam ownership, the kinds of hazards that may actualize these risks and the reasons why dams fail. Risk related to possible dam failure is continuing to grow higher for three obvious reasons: 1) there is an increasing risk due to our aging dams; 2) because people have been allowed to settle below dams in potential inundation zones; and 3) because new dams are being built in less than ideal sites. Other elements of risk include natural phenomena such as floods, earthquakes and landslides. These hazards threaten dam structures and their surroundings. Floods that exceed the capacity of a dam's spillway and then overtop the dam or abutments are particularly hazardous, as is seismic activity that may cause cracking, seepage, or complete collapse of the dam. Similarly, debris from landslides may block a dam's spillway and cause an overflow wave that could erode the abutments and ultimately weaken the structure.

Since the terrible national disaster of 9/11/01, there has been a heightened concern regarding possible terrorist threat or sabotage to our dams or other water facilities. Although earthen dams in general are less vulnerable to terrorist bombs or other modes of attack than other types of dams, due to their massive size and ductile performance, there are still certain things that can be done to protect these structures. Suffice it to say that the District has performed or will perform the necessary steps to protect its dams.

The International Commission of Large Dams (ICOLD) has determined that the three major categories of dam failure are 1) overtopping by flood, 2) foundation defects, and 3) piping (progressive internal erosion by seeping water). For earthen dams, the major reason for failure is piping or seepage. For concrete dams, the major reasons for failure are associated with foundations. Overtopping has been a significant cause of dam failure primarily in cases where there was an inadequate spillway. As far as the District dams are concerned, all spillway capacities were independently evaluated for probable maximum flood (PMF) conditions as a part of the 1980s National Dam Safety Program. As a result of these studies, performed by DSOD under contract to the Corps of Engineers, the spillway capacity at Anderson Dam was significantly increased in size (from 20,000 cfs to 60,000 cfs) in 1987. In a similar fashion, following the 1989 Loma Prieta earthquake, it was discovered that the crest of Lenihan Dam had lost a significant portion of its freeboard, due in part to self-weight consolidation since construction and a loss of not quite a foot due to earthquake-induced shear strain during the earthquake. DSOD requested that the District reinstate the lost freeboard, and to also upgrade the spillway capacity to PMF conditions. The resulting freeboard restoration work was completed in 1996.

The concept of using potential failure modes has always been a part of designing a monitoring program for a dam. However, in recent years, the federal agencies entrusted with national dam safety (USBR, COE, and FERC) have embraced the concept of Potential Failure Mode Analysis (PFMA) on their dams. It appears that it is only a matter of time before such methods will be mandated on other dams under their license (such as Anderson Dam). The FERC has established a draft Performance Monitoring Program (PMoP) for the purpose of establishing guidelines for monitoring the performance of dams and guidelines for safety reports and analysis. FERC has already completed pilot studies on several existing dams to evaluate the effectiveness of the PMoP approach. It is clear that it is only a matter of time before such methods will become accepted practice for dam safety.

3.4 Justification and Development of a Dam Safety Program

Recognition of the causes and possible impacts of dam failure points out the need for a program to ensure dam safety. Such a program should be based on a safety evaluation to determine a dam's structural and operational safety. The evaluation should identify problems and recommend instrumentation and monitoring requirements, remedial repairs, operational restrictions and modifications, or further analyses and studies to determine solutions. For District dams, the original construction was completed well before any formal dam safety program was in force. Nevertheless, as briefly discussed in Chapter 1, the District dams were well designed and constructed and have performed exceptionally well over the life of the structures. The major exception to this performance history has been the series of outlet pipe problems that plagued Lenihan Dam. Formal dam safety inspections have been carried out for all District dams dating back to the mid-1970s. Also in the mid-1970s, the District began a systematic evaluation of the static and dynamic (seismic) stability of its dams. Each of the dams has had static stability analyses performed to assess their safety under normal operating conditions, and two dams (Lenihan and Almaden) have had detailed seepage analyses performed. Lenihan, Stevens Creek, Guadalupe, Calero, and Almaden Dam have had rigorous full-scale dynamic stability analyses performed to assess the probable performance of the dam during major earthquake loading scenarios. Anderson Dam has had a detailed simplified dynamic analysis performed to assess the potential impact of a major earthquake occurring on the nearby Calaveras fault. A discussion of the detailed technical issues and types and recommended frequencies of the various static and dynamic analyses associated with the Dam Safety Program is presented in Appendix E.

A safety program comprises several components that address the spectrum of possible actions to be taken over the short and long term. Development of a safety program involves a phased process beginning with collection and review of existing information, proceeding to detailed inspections and analyses, and culminating with formal documentation. For District dams, the primary problems identified in recent dam safety inspections include;

1) need for new instrumentation;
2) lack of performing routine maintenance at our dams; and
3) need for new seepage collection and monitoring systems at Almaden and Calero Dams. In accordance with appropriate District design procedures, this work will require the completion of an Engineers Report and the preparation of appropriate CEQA documents (i.e., preparation of a Categorical Exemption, Negative Declaration, or EIR).

With regard to instrumentation, Lenihan Dam has been provided with a modern automated instrumentation system. Appendix G presents the preliminary instrumentation design for the other ten dams to bring them to modern dam standards. The significant problems include the aging outlets and continued reservoir sedimentation. These issues are being addressed by capital projects at Lenihan Dam, Almaden Dam, with similar projects soon to follow at Calero Dam and Guadalupe Dam.

As discussed above, the inspection program for continuing safety evaluations establishes the condition of the dam and is a major source of the information necessary for determining specific actions to be taken regarding repairs, operations, and monitoring. Dam safety inspection programs are periodic, recognizing the need for continued vigilance. Also, it is very important for District staff involved with all aspects of dam safety to maintain on-going training and education in both regular dam maintenance and new methods of engineering analysis of dams.

3.5 Environmental and CEQA Concerns for the DSP

The District's dams, by virtue of their locations, tend to be in natural, aquatic and vegetated environments that are natural habitat for wildlife, some of which are federally or state protected. Indeed, several of the District dams are habitat to threatened and endangered species such as the red-legged frog and the checkered butterfly, and the streams below a number of our dams are now considered to be habitat to salmon and steelhead. Many of these areas represent habitat improvement due to our own making. For example, the "wetlands" that exist downstream of Calero Dam is actually water that got trapped in former borrow areas that were used to construct the dam. They were created as a direct result of the historical releases of water from the outlet pipe and the construction of the Almaden Valley Pipeline which redirected where the discharges from the dam were to occur.

Typical dam safety and dam maintenance practice is to maintain the entire structure, including appurtenances, and a modest setback, free of unwanted vegetation and growth; basically, maintaining the structures in their "as-built" condition. This is necessary in order to visually inspect for unwanted seepage emerging from the contact between the dam and the abutments. The typical exception is that grasses are often used as an erosion control feature, typically on the downstream face of the dam. Thus, the District is in the difficult position of trying to maintain the as-constructed condition of the dam, in the setting of a relatively pristine and sensitive environment.

Within the past year or so, because of the environmental setting of these structures, the District environmental staff has taken the position that we should complete CEQA documents and obtain permits for most of our routine maintenance activities, even though some of these activities may appear to be relatively benign. This typically requires the filing of, as a minimum, a Categorical Exemption (CE), and possibly a Mitigated Negative Declaration (MND), or EIR for each activity. Currently, due to the presence of endangered species, the environmental consideration, possibly including an EIR, is being applied to the geotechnical/geological investigation required for design. Once the design is completed, the environmental permitting process will be repeated for the overall construction project. It must be understood that this process is unprecedented for dam maintenance issues and takes an enormous amount of time and resources.

The Environmental staff at the District has advised the Dam Safety Section that the grouping of typical activities into a Programmatic EIR for dam maintenance, similar to the Stream Maintenance Program (SMP), is the best way to streamline the environmental permitting process. The District is in the initial stages of budgeting for a project to prepare such a document, but it is not expected to be completed for at least 3 to 4 years. In the meantime, the prolonged and piecemeal process of activity-by-activity environmental clearance is still being evaluated.

Another environmental consideration exists with the planned implementation of the critical Dam Instrumentation Program (DIP) component of the DSP (see Appendix H). The responsibility for the final design, development of plans and specifications, and construction of the project will be with the Capital Program Services Division (CPSD), including obtaining the appropriate environmental clearance.

3.6 Dam Maintenance Oversight

As discussed earlier in this chapter, dam inspections and dam maintenance are perhaps two of the most significant elements of the DSP. The District has historically had a very effective, self-regulating program for both of these vital dam safety endeavors, which we propose herein to continue and, if possible, upgrade. The District has been rewarded for it's effectiveness in the past in these areas, with an excellent reputation with the two significant state and federal regulatory agencies, DSOD and FERC, respectively. However, within the past couple of years, the District's effectiveness in dam maintenance has been severely impacted due in part to: 1) the need for new environmental permits; and, 2) less effective oversight and tracking of the dam maintenance function. The latter problem apparently has occurred due to recent District reorganizations. As a result of the difficulties surrounding routine maintenance of the District's dams, its responsiveness to regulatory agency requests has been significantly impaired. It also appears that the hard-earned reputation and goodwill that the District has earned from DSOD and FERC is gradually diminishing, if not already seriously jeopardized. The causes for this trend, along with our evaluation of the current condition, and our proposals to correct the situation are the subject of this section.

Staffing and budgeting are significant problems both from an operational and environmental point of view. The budget for performing dam maintenance activities lies with the Utility Maintenance Unit; unfortunately however, no one in the unit is assigned the responsibility for coordinating and tracking the activities and no environmental planner is specifically assigned to assist with what will no doubt be a time consuming task over the next few years. The task of tracking and coordinating dam maintenance has historically been managed within the District by the Operations, Planning and Analysis Unit, with oversight from the Dam Safety Section of the Water Utility Engineering Unit. It is recommended that for at least the next 5-year period that the oversight responsibility of the dam maintenance program be assigned to the Dam Safety Section. This will require the addition of at least one fulltime staff position who would be responsible to track and oversee maintenance and to assist and facilitate environmental planning of dam maintenance issues. A 5-year period should be sufficient to implement a Programmatic EIR for Dam Maintenance, complete the seepage mitigation requirements at Almaden and Calero Dams, and once again reestablish the routine maintenance function for District dams back to a more routine program.

Completion of actual dam maintenance would normally be performed by the appropriate Watershed Field Operations Unit, the Vegetation Management Unit, or the Utility Maintenance Unit, but they are basically unable to perform the work without the appropriate environmental clearance. These units do not have the resources to provide the necessary environmental support. Environmental support should come from the staff of County-wide Watershed Management. An appropriate budget allocation for the type of environmental effort that is now being considered and advocated regarding dam maintenance was not envisioned or anticipated in the past. As a result, maintenance activities on the dams have slowed down dramatically. Within the past two years, annual DSOD and FERC inspections of District dams typically resulted in a list of requested or required corrective repair or maintenance activities. Since the Spring 2000 inspection, the list of activities has been piling up due to pending environmental permits and ineffective oversight. During the Fall 2002 safety inspections, DSOD told District staff that they may be forced to use their regulatory authority to limit the District's use of its facilities until the required maintenance or safety issues are addressed. Most likely, such restrictions would be imposed on a step-by-step basis. If such actions were to be taken by DSOD, and based on observations with other dam owners, such restrictions could get progressively more aggressive and remain in effect until DSOD has visual evidence that the dam owner has addressed their particular safety concern.

It is worth mentioning, that the unpredictable occurrences of moderate-sized earthquakes could also result in a significant amount of corrective repairs at the District dams. Moderate or even minor earthquake damage may likely be viewed as a safety issue with the regulatory agencies, requiring immediate repair, or the possible consequence of imposition of severe operating restrictions. The District's ability to respond rapidly in such instances, given the conflicting requirements of the environmental regulations, could be a concern and should definitely be considered in any future Programmatic EIR for dam maintenance.

Partially as a result of maintenance lapses, there has been additional unplanned habitat growth within the limits of the as-constructed dam facilities, further exacerbating the problem of performing dam maintenance. One reason for the encroachment is uncontrolled leakage through several of the dams. The resulting wet areas have encouraged rapid growth of vegetation, and migration of protected species into the area. Once allowed to take hold, these areas are no longer able to be maintained without even more extensive environmental consideration (i.e., mitigation).

Coincidently, the uncontrolled seepage from the dam can be a safety concern, and the State regulatory agency, DSOD, has directed the District to control and monitor (not eliminate) the seepage problems. Therefore, the District is in the planning stages of a project to collect, control, and monitor seepage at Almaden and Calero Dams. These projects are in the environmental clearance stage for the field investigation, leading to conceptual design. The actual construction projects will require a separate environmental clearance. The cost and time required for environmental clearance for these projects may have been significantly underestimated in the past.

3.7 Inspection Guidelines

An effective inspection program is essential to identify problems and to provide for safe maintenance of a dam. The inspection program should involve three types of inspections: (1) periodic technical inspections, (2) periodic maintenance inspections, and (3) informal observations by District personnel as they operate the dam. The remainder of this section deals with these types of inspections.

3.7.1 Technical Inspections

Technical inspections of dams involve a team of people with specialized expertise in engineering, geology, and operations. As mentioned previously, technical inspections are currently being performed at all District dams at least once per year collectively by District and DSOD personnel and an additional FERC inspection of Anderson Dam every year. These inspections include a detailed visual inspection of the dam and appurtenant facilities. The visual inspection should include a detailed inspection of the exposed section of the upstream face, crest, downstream face, and abutment groins to note any cracking, settlement, seepage, or other potentially adverse development. Observations are recorded and photographs are taken, preferably at identified locations for comparisons with previous circumstances. During recent safety inspections, for example, it was noticed that several of the spillways at District dams have begun a movement heretofore not experienced. This movement appears to have begun following the 1989 Loma Prieta earthquake and may represent tectonic movement or adjustments to locked-in stresses generated during the earthquake. At Coyote Dam, the movement gradually created unacceptable cracking of the spillway floor slab that was subsequently repaired in 2001.

The in-pipe inspection of outlet pipes, by either remote vehicle or manual inspection, is difficult because most of these facilities are under continuous operations. Any such inspection of these facilities requires extensive coordination and planning. Outlet valves and gates should be periodically exercised to full open conditions on an annual basis to insure functionality and proper operation. Outlet pipes should be inspected once per five years to insure that the pipe has not deteriorated and can continue to be safely used for full discharge potential. This is typically done using Remotely Operated Vehicles (ROV) equipped with a video camera. It is important that the ROV be extended all the way to the outlet valve to visually check for leaks or other signs of malfunction. Follow-up outlet inspections by staff may be required if the ROV inspection either reveals problems or does not provide adequate information. Such ROV inspections for outlet pipes should be programmed on a regular 5-year cycle for all District dams. A history of outlet pipe inspections completed at District dams over the last decade is presented on Table 3-1.

It is very important to carefully document all inspections, both in written and visual format. Photographs should be taken from the same locations on the dam or abutments, if appropriate, to show the time history or progression of a particular event. Sometimes, more elaborate measures are necessary. For example, at Calero Dam, a portable GPS device is used to monitor the location and limits of the growth of phreatophytes (water loving plants) on the downstream face and near the downstream toe in order to accurately track the location of emerging seepage on the downstream face of the dam. By taking GPS reading at different times of the year and/or at various reservoir levels, it is possible to document the lateral extent of seepage rather than relying on memory. It should be noted that DSOD inspectors prepare a formal report following each inspection. This report should be carefully checked with the District report to insure consistency. It is recommended that the central filing system be modified to accommodate the new measures for the Dam Safety Program discussed herein. Prior to implementing this recommendation, however, it is recommended that an in-depth assessment and evaluation be performed by records management personnel to determine the practicality of developing such a independent filing system.

3.7.2 Post-Earthquake Inspections

The District's Emergency Operations Manual (SCVWD, 1976) covers earthquake emergency procedures. The general guideline stipulates that…"when a damaging earthquake occurs, those District facilities that present the greatest potential danger to the public will be inspected first, with those of less potential danger inspected later." In anticipation of this need, the District has developed extensive post-earthquake inspection procedures for its dams. The program is described in detail in Appendix D.

A small earthquake ($M_L = 4.9$) occurred southwest of Gilroy on May 13, 2002. Since the earthquake was originally estimated at $M=5.2$, the post-earthquake dam inspections were automatically triggered. The closest District dam to the epicenter was Uvas Dam at 8.4 miles. Anderson Dam is located 13.9 miles from the epicenter. Due to the relatively close proximity of Anderson Dam to the epicenter, FERC requested that the District inspect the dam with a FERC inspector. It was decided to complete a post-earthquake inspection report for this inspection. A copy of the inspection report is presented in Appendix D and represents a model for future post-earthquake inspection reports.

Table 3-1
OUTLET PIPE INSPECTIONS
1992-2002

NAME OF DAM	1992	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002
Alamden								January 11-15, 1999 Sedimentation Investigation Core samples of sediment ROV Inspection of outlet pipe Buoy attachment to intake		October 17, 2001 Diver inspected outlet pipe Underwater Resources supported by District crews.	
Anderson											
Calero			November 22, 1994 Inspection of intake structure by North Coast Divers supported by District crews.						June 20, 2000 ROV Inspection of outlet pipe Interspect Remote Systems supported by District crews.		
Chesbro											
Coyote						October 14, 1997 DSOD Annual inspection and boring on spillway floor to determine cause of delamination of concrete. Inspected outlet tunnel		April 16, 1999 DSOD Annual inspection Rotated U/S and D/S Valves Inpected Outlet Tunnel	May 4, 2000 DSOD Annual inspection Rotated U/S and D/S Valves Inpected Outlet Tunnel October 31, 2000 DSOD Annual inspection Rotated U/S and D/S Valves Inpected Outlet Tunnel	September 13, 2001 DSOD Annual inspection Rotated U/S and D/S Valves Inpected Outlet Tunnel	
Guadalupe			November 22, 1994 Inspection of inlet tower						June 22, 2000 ROV Inspection of outlet pipe Interspect Remote Systems supported by District crews.		November 19, 2002 Diver Inspection of outlet pipe Pan Marine performed inspection supported by District crews.
Lexington						February 24, 1997 Video inspection of outlet pipe reveals 2 collapsed sections. March 1, 1997 Followup inspection of outlet pipe	August 25, 1998 Divers inspection of outlet pipe reveals 1 collapsed section.		June 20, 2000 ROV Inspection of outlet pipe Interspect Remote Systems supported by District crews and DSOD.		
Stevens Creek											
Uvas											
Vasona											
Rinconada Reservoir											

Prepared by Joe Aguilera
3-15-03

3.7.3 Maintenance Inspections

A good maintenance program will protect a dam against deterioration and prolong its life. A poorly maintained dam will deteriorate and could fail. Nearly all the components of a dam and the materials used for dam construction are susceptible to damaging deterioration if not properly maintained. A good maintenance program provides not only protection for the owner, but for the general public as well. Furthermore, the cost of a proper maintenance program is small compared to the cost of major repairs or the loss of life and property. The District has had a basic maintenance program, based primarily on systematic and frequent inspections, in force since the dams were completed, or in the case of Chesbro and Uvas Dams, since taking over their operations. Maintenance inspections should be done monthly and after major flood or earthquake events. During each inspection, a checklist of items calling for maintenance should be used.

Maintenance inspections should be performed more frequently than technical inspections in order to detect, at an early stage, any detrimental developments in the dam or appurtenant structures. These inspections involve assessment of operational capability as well as structural stability. Unfortunately, maintenance inspections of District dams have not been carefully documented in the past. Recently, the maintenance for most District facilities has been placed on the MAXIMO program, a versatile computerized maintenance management software program which is capable of creating, tracking, and automatic scheduling of normal maintenance requirements. Dam safety personnel should be trained in using the MAXIMO program and become involved with establishing what maintenance items are placed on the original list. A unique list of maintenance items should be established for each dam, although many items may be similar. The Dam Safety Section personnel should be responsible for the periodic monitoring of the items required for maintenance and have access to the MAXIMO program to independently verify the progress of the planned maintenance program. Utilizing the MAXIMO program will be especially helpful to the Dam Safety Section in tracking scheduled maintenance work since this work is planned and executed by a number of other units within the District.

The keeping of maintenance records is also critically important for the dam safety program. Current record keeping of completed maintenance at the dams is not rigorously recorded. Someone from the Dam Safety Section should be responsible keeping maintenance records at all District dams. The justification for better maintenance records is that if problems were to develop at a dam, the ability to go back and search records is a potentially valuable piece of information as to what may have led up to the problem in the first place. It would appear that MAXIMO may be the ideal tool to keep such records. The Dam Safety Section should explore such possibilities and implement a method of systematically documenting all planned and unplanned maintenance activities.

3.7.4 Informal Inspections

The third type of inspection is actually a continuing effort by operations and maintenance personnel performed in the course of their normal duties. In order to gain additional valuable information, all operations and maintenance personnel who visit the dams should receive specialized training from dam safety personnel on what to look for when visiting any dam. These personnel should also receive other formal training from dam safety training experts. Once trained as to what to look for, every on-site operations and maintenance personnel can provide invaluable assistance in spotting problems before they become significant. Nevertheless, there needs to be an easy and systematized way for operations and maintenance personnel to convey their concerns, should they have any. This concept of an easy debriefing of maintenance personnel should be further considered by Dam Safety personnel.

3.8 Instrumentation and Monitoring Guidelines

Instrumentation of a dam furnishes data to determine if the structure is functioning as intended and provides a continuing surveillance of the structure to warn of any unsafe developments or trends. Means and methods available to monitor physical phenomena that can lead to a dam failure include a wide spectrum of instruments and procedures ranging from very simple to very complex. Any dam safety instrumentation program must: 1) involve proper design consistent with other project components, 2) be based on prevailing geotechnical and geological conditions at the dam, and 3) include consideration of the hydrologic and hydraulic factors present both before and after the project is in operation. Instrumentation designed for monitoring potential deficiencies at existing dams must take into account the threat to life and property that the dam presents. Thus, the extent and nature of the instrumentation depends not only on the complexity of the dam and the size of the reservoir, but also on the potential for loss of life and property downstream. A well-designed instrumentation program should involve instruments and evaluation methods that are as simple and straightforward as the project will allow.

Moreover, the dam owner should make a definite commitment to a continuing monitoring program. If the program is not continuing, the installation of instruments and procedures will be wasted. Obviously, the involvement of qualified personnel in the design, installation, monitoring, and evaluation of an instrumentation system is of prime importance to the overall success of the program.

Instrumentation and proper monitoring and evaluation are extremely valuable in determining the performance of a dam. Instrumentation can provide to following information:

- Warning of a developing problem
- Definition and analysis of a problem
- Proof that behavior is as expected
- Remedial action performance evaluation
- Alarm and notification of emergency response personnel

3.8.1 Special Considerations for Surveying

Survey techniques have significantly changed within the past decade, especially with the advent of the Global Position System (GPS) method which was introduced at the District in January 1994. Although the principal method used in completing the survey at a dam is the tried and true method of station and offset, in fact, one or two points at each dam are tied to a set of base monuments within Santa Clara Valley. This work is done using GPS methods and it is then possible to tie the GPS coordinates to all subsequent monitoring points.

Recently, District technical staff has noticed a disturbing drift in the GPS determined locations at some of our dams. It turns out that the likely reason for this "drift" is the ever constant creep that is occurring adjacent to the major faults in the Bay Area (caused by continental drift) thereby causing the base monuments to move, and yet the District is currently using them as though they have not. As a result, the District is considering implementing a new Geodetic Control Maintenance Program (GCMP), under the direction of the Land Surveying and Mapping Unit, that will provide the District with state-of-practice capabilities in 24/7 GPS monitoring.

3.8.2 New Methods of Surveying Irregular Locations

New methods of survey have been recently developed for irregular surfaces that use a technique of multi scanning with lasers. The District has purchased such a unit manufactured by Cyra Technologies, Inc. and referred to as the Cyrax 3D Laser Scanner. Although the unit was primarily purchased for helping to develop as-built topology for District water treatment plants, it would appear that the unit can also be used to aid in developing topographic maps for difficult sites that are virtually impossible to survey using conventional techniques. The unit captures 3D surface geometry of complex structures and various terrain sites. The scanning system consists of a laser scanner, instrument tripod and optional targets for position refinement. The scanner is oriented towards the project site and the area limits and scanning density are programmed into the scanner. The Cyrax unit then completes an auto-scan of the site based on the selected options. Complete surface geometry of exposed surfaces are remotely captured in the form of dense, accurate "3D point clouds". The scanner can be rotated or moved around the site to capture entire scenes. Scanned surfaces can be anything from erosion sites, bridge details, or complicated pipe details.

Based on a preliminary assessment of the Cyrax unit, its use in monitoring potential deformations or other types of erosion is very encouraging. It is strongly recommended that it be used to at the following facilities:

- The existing eroded ground surface conditions adjacent to the Lenihan Dam spillway discharge area. This area has been experiencing serious erosion for many years and it is virtually impossible to complete the required survey without suspending the surveyors from the spillway. The principal concern for this area is to determine whether the erosion is undermining the safety of the spillway.

- The area downstream of the Anderson Dam spillway falls area. This area has experienced significant erosion during periods of heavy spillway discharge. It is important to determine the approximate rate of erosion in this area to determine whether slope reinforcement or some other form of erosion protection is warranted.

- Coyote Dam spillway slab. This area has experienced minor, but virtually continuous, slow creep since the 1989 Loma Prieta earthquake. The amount of time to survey this area by conventional methods is significant.

- Several concrete wall panels of the Calero Dam spillway in the downstream right wall area. Several wall panels in this area have experienced movement and bulging and apparently are the result of minor slope movements adjacent to the right side of spillway. The area has had at least one repair.

It is recommended that the Cyrax unit be used to generate a 3D model of each of these areas on an annual basis for at least several years. The results can then be compared in order to determine a time rate of erosion or movement. Based on the results, a more convenient interval of readings can be established. It is further recommended that an evaluation be performed to determine whether the Cyrax unit could be used to establish the pre-earthquake condition/location for the upstream and downstream areas at all District dams, including spillways. The principal objective of this evaluation would be to determine if the Cyrax unit could be used to document existing pre-earthquake conditions that could then be compared with post-earthquake conditions in order to determine a more exact assessment of earthquake-induced deformation.

3.9 Emergency Response

Although most dam owners have a high level of confidence in the structures they own and are certain their dams will not fail, history has shown that on occasion dams do fail and that often these failures cause loss of life, injuries and extensive property damage. All dam owners should prepare for this possibility by developing an emergency response plan which provides a systematic means to accomplish the following:

- Identify emergency conditions threatening a dam;
- Expedite effective response actions to prevent failure; and
- Reduce loss of life and property damage should failure occur.

The dam owner is responsible for preparing a plan covering these measures and listings the actions that the owner and operating personnel should take in the event of an emergency. They should be familiar with the local emergency management officials and agencies responsible for warning and evacuating the public. It is important that dam owners coordinate fully with others who are concerned with public safety. Emergency plans will be more effective if they integrate the actions of others who can expedite response. People and organizations with whom the dam owner should consult in preparing an emergency action plan must include numerous local participants, and state and federal agencies.

An essential part of the emergency response plan is a list of agencies/persons to be notified in the event of a potential failure. Possible inclusions for this list should be obtained from and coordinated with local emergency management agencies. These are key people or agencies who can activate public warning and evacuation procedures or who might be able to assist the dam owner in delaying or preventing failure.

Certain key elements must be included in every notification plan. Information about potential inundation (flooding) areas and travel times for the breach (flood) wave is essential. Inundation maps are especially useful in local warning and evacuation planning. It should be noted that California State Law require that the District develop and maintain inundation maps for all dams. FERC has special requirements for Anderson Dam. The State Office of

Emergency Services (OES) has received inundation maps for all dams owned and operated by the District. This concept of inundation maps is further discussed below in Section 3.10.

The District has a full time professional emergency management program (Office of Emergency Services) charged with the responsibility to train all emergency operations personnel, in addition to activation and planning for all levels of emergency. A fully equipped Emergency Operations Center, with sufficient room for up to 25 EOC personnel to respond to emergencies, is maintained as a separate facility with independent power generation at District facilities. The District has adopted the Standardized Emergency Management System (SEMS) to respond to any identified emergency. SEMS is the system required by State Law for managing response to multi-agency and muti-jurisdiction emergencies in California. The Emergency Operations Center at the District consists of 125 District personnel, all of whom receive regular training in SEMS and the Incident Command System (ICS).

A part of the Emergency Response Plan that has been implemented at the District is earthquake preparedness. An in depth discussion of the considerations for this most important program is presented in Appendix D.

3.10 Operation Plan Guidelines

An operation plan addresses the water supply and flood control functions of the dam and reservoir. Establishing an operations procedure or plan calls for the following:

- Detailed dam and reservoir physical characteristics data;
- Descriptions of dam components;
- Determination of the "standard of care" for each facility;
- Operations instructions for operable mechanisms;
- Inspection instructions;
- Instrumentation and monitoring guidelines;
- Maintenance operations guidelines;
- Temporal water storage and flood response;
- Emergency response operations guidelines;
- Bibliographical information.

A schedule should be established, and tied to the MAXIMO system, to include both day-to-day tasks and tasks performed less frequently throughout the year. The schedule formalizes inspection and maintenance procedures so that even an inexperienced person can determine when a task is to be done.

3.11 Measures to Reduce the Consequences of Dam Failure

The dam owner can use or implement a variety of other measures that will help reduce the consequences of dam failure. Some land use measures instituted by governments represent better means of mitigating future disasters. If people are restricted from living in inundation zones, then safety is radically improved; however, such measures are not practical for the

District due to the existing land use below the dams in question. Instituting land use measures represents one of the most effective ways to save lives and property over the long term, but such steps are not always acceptable to governments.

Other measures such as land use, public awareness and preparedness planning, are essentially controlled by local governments. Therefore, dam owners would be wise to encourage as strongly as possible awareness and action in the public sector. Finally, they may also wish to hire consultants from the private sector when the information needed for prudent decisions exceeds their own expertise.

One of the elements of dam safety and the emergency response plan previously discussed is the development of flood inundation maps in the event of a dam should fail. Such maps were developed at the District as a requirement of the State Office of Emergency Services (OES). An examination of the existing inundation maps suggests that there is a need to update the current maps, as is currently being done for Anderson Dam. As a part of this Dam Safety Program, it is recommended that the District's Office of Emergency Services, along with appropriate dam safety personnel, meet with appropriate local and state agencies every five years to review the adequacy of inundation maps and emergency action plans for all District dams. This activity should probably be scheduled for perhaps just two to three dams per year, rather than trying to do all 11 dams once per five years.

Chapter 4

Staffing and Cost Estimates for Implementing the Dam Safety Program

4.0 STAFFING AND COST ESTIMATES FOR IMPLEMENTING THE DAM SAFETY PROGRAM

4.1 Introduction

This chapter presents cost and staffing estimates for implementing the Dam Safety Program. This report has been developed to specifically aid current and future District staff in how to assess, maintain, and report on future dam safety issues. It is also intended for use by the current or future CEO in their oversight responsibility to justify future development of staff and expenditure of funds related to implementing and maintaining the DSP. The justification for such action as may be warranted and appropriate is founded in the governance policies of the District's Board adopted in 2000, wherein they issued directives to the CEO/GM as follows:

Policy E-1:

"The mission of the District is a healthy, safe, and enhanced quality of living in Santa Clara County through the comprehensive management of water resources in a practical, cost-effective, and environmentally-sensitive manner."

Policy EL-1.6:

"The CEO/GM shall not allow corporate assets to be unprotected, inadequately maintained or unnecessarily risked. Further, without limiting the scope of the foregoing by this enumeration, he or she shall not:

3. *Subject plant and equipment to improper wear and tear or insufficient maintenance.*
4. *Unnecessarily expose the organization, its Board or staff to claims of liability.*
5. *From Executive Limitations: The CEO shall develop a systematic means of monitoring and reporting accomplishments."*

4.2 Instrumentation Schedule and Estimated Cost

The Dam Instrumentation Program (DIP), which will consist of installing new instruments and automation for ten District dams is presented in detail in Appendix H. The dams include Almaden, Anderson, Calero, Chesbro, Coyote, Guadalupe, Stevens Creek, Rinconada Treated Water Reservoir, Uvas, and Vasona Dams. The installation of instruments at these dams is at the heart of the Dam Safety Program and will provide the District with assurance that all District dams are up to the current state-of-practice with regard to modern earth dam instrumentation and automation. In order to implement the desired instrumentation plan, the District could have contracted with a consultant to complete the requisite design. This would have required that the consultant thoroughly familiarize themselves with essentially the full 64-year history of dam construction, operation, and performance issues for each dam. In lieu of hiring a consultant, it was decided that the current staff of the Dam Safety Section of the Water Utility Engineering Unit could adequately and efficiently complete the work. This decision was easily justified since the current Dam Safety Program staff has a extensive record of having dealt with safety-related issues for all District dams, and since that staff had the opportunity and responsibility to develop in-depth dam surveillance reports for all District dams for submittal to DSOD and FERC during the last two years. As a part of this latter activity, the Dam Safety staff had the opportunity to complete a thorough review of the design, construction, and major repairs for all of the District's dams.

4.2.1 Instrumentation Schedule

The proposed schedule for developing the final design and installation of the recommended instruments is presented in Figure 4-1. As shown, the required work associated with all aspects of implementing the instrumentation plan is expected to take about 6 years. The actual schedule will be as approved by the CEO and the Board. The work associated with the final design and actual installation of the instrumentation planned for the Dam Instrumentation Project requires Board approval. Prior to seeking approval for specific work elements, it will be necessary for the District staff to complete other related studies that are typically required for such major programs. This work includes the completion of a decision document (Engineers Report) that fully describes and evaluates the proposed approaches and alternatives, and documents the decision process to proceed with the Dam Instrumentation Project as currently envisioned. Of particular concern for the preparation of the Engineers Report will be the selection of the preferred type of instrumentation, as well as the currently envisioned automation equipment. As more fully described in Section 3.5 of this report, in addition to the preparation of the decision document, CEQA documentation will need to be prepared for the Dam Instrumentation Project as well as needed maintenance and repair of the existing downstream seepage collection systems for Almaden, Calero, Guadalupe, and Uvas dams.

The priorities associated with completing the required instrumentation work have been prepared and are presented in Table 4-1. As shown, it has been assumed that the final design and installation of instrumentation will be completed in groups of two dams each. The schedule could be significantly shortened if it were decided that there could be cost savings if the grouping of dams for final design and installation were expanded to three or four dams each. The priorities have been established with regard to both risk and instrumentation adequacy. Depending on the timing of the planned work, and the possible continued deterioration of existing seepage conditions near the downstream toe of the four dams mentioned above, it may be necessary to alter the planned sequence, especially for the planned seepage modifications, or to complete the seepage remediation separately.

It is intended to complete all of the design and development of the plans and specifications with present District staff. The project would be assigned to the Capital Services Improvement Division for management and execution, with the Water Utility Engineering Unit providing expert consultation regarding final design, and the development of final Plans and Specifications. This is similar to the approach and successful completion of the Phase B Instrumentation project for Lenihan Dam that was finished within time and budget. The District will also provide construction management during actual installation of the instrumentation. The WUE Unit will provide staff or a consulting geologist to develop detailed geologic logs and driller oversight as needed.

4.2.2 Instrumentation Costs

The estimated costs for the instruments associated with the Dam Instrumentation Plan are presented on Tables 4-2 and Table 4-3. Table 4-2 presents the estimated costs to purchase and install the instruments. The cost estimate is based on our recent experience with instrumenting Lenihan Dam. Unit prices for the Lenihan Dam Phase B work were developed on a basis of cost per foot, cost per hole drilled, and cost per instrument installed. These data were then applied to the other 10 dams using judgment as to which unit price was appropriate depending on the scope and number of instrumentation at each facility. The estimate includes the cost of instrumentation (piezometers, inclinometers and accelerometers, if appropriate), installation of instruments and cabling, and the automation equipment and hookup. Table 4-3 presents the total estimated cost to implement the Dam Instrumentation Project. In developing theses costs, the cost for final design and the development of plans and specifications would be about 20% of the total purchase and installation costs. The construction management was estimated at 10% of the instrument costs. Finally, a 15% contingency was added for all project elements.

The cost estimate is presented on Table 4-2 shows that the total construction cost for the automated instrumentation system for all 10 dams is estimated to run about $3.5 million, or an average of about $350,000 per dam. The highest cost estimate is about $700,000 for Anderson Dam, and the lowest is about $60,000 for Vasona Dam. As shown in Table 4-3, the total estimated cost to implement the Dam Instrumentation Project is about $4.3 million. The current plan is that it will take about 6-7 years to complete the work. This amounts to an average expenditure of about $800,000 per year. It may be appropriate from an overall design and construction management standpoint to expedite the DIP and have it completed within a 2-3 year-period. Regardless of the final schedule, the final design and development of the plans and specs will be completed by the Capital Program Services Division with technical oversight and consultation provided by the Dam Safety Group. The recommended DIP will elevate the instrumentation of District dams to an appropriate level of confidence and is considered to represent the current state-of-practice in the area of dam safety. The instrumentation program is designed to serve the District needs in dam safety for at least the next 20 years or longer.

4.3 Personnel Qualifications and Responsibilities

This section presents an estimate of the staffing requirement for the Dam Safety Program. There are a number of work efforts that must be adequately addressed to provide for dam safety, including:

- Adequate periodic inspection;
- Preventive/scheduled maintenance;
- Adequate instrumentation;
- Timely corrective maintenance;
- Adequate static and dynamic stability analyses and seepage analyses to assess the overall safety of each dam;
- Capital projects where needed;
- Continued evaluation of data from inspections and instrumentation, with resulting yearly surveillance reports;
- Post-earthquake inspections;
- Site specific Emergency Action Plans.

Table 4-1 Prioritized Schedule for New Instrumentation

Dam	Types of Proposed Instrumentation	Required/Anticipated Ancillary Work
Almaden	Vibrating wire gauge piezometers, open well piezometers, accelerometer	Seepage collection system at toe of dam and installation of new D/S weirs, tree removal and cleanup from D/S berm
Guadalupe	Vibrating wire gauge piezometers, open well piezometers	Seepage collection system at toe of dam and installation of new D/S weirs
Anderson	Vibrating wire gauge piezometers, open well piezometers, accelerometer, automated inclinometer	Installation of monitoring well about 50 feet from downstream toe in County Park parking strip.
Chesbro	Vibrating wire gauge piezometers, open well piezometers	None
Coyote	Vibrating wire gauge piezometers, open well piezometers, accelerometer	None
Stevens Creek	Vibrating wire gauge piezometers, open well piezometers, accelerometer, automated inclinometers	None
Calero	Vibrating wire gauge piezometers, open well piezometers	Seepage collection system, new D/S weirs in right abutment
Uvas	Possible restoration of existing open well piezometers with vibrating wire gauge piezometers plus installation of new vibrating wire gauge piezometers	Seepage collection system, new D/S weirs.
Rinconada	Vibrating wire gauge piezometers	None
Vasona	Possible restoration of existing open well piezometers with vibrating wire gauge piezometers plus installation of new vibrating wire gauge piezometers	None
Lenihan Dam	No new instrumentation required	Repair of bike path erosion, installation of new drainage ditches, and cleanup of brush from downstream toe area near outlet.

Table 4-2 - Cost Estimate for Instrumentation and Automation at 10 District Dams

Dam Name	Estimated Cost of Instruments Plus Installation	Estimated Cost of Automation Equipment	Total Cost To Purchase and Install Instrumentation (No Plans & Specs)
Almaden	$ 233,500	$ 50,000	$ 285,500
✓ Anderson	$ 551,000	$ 150,000	$ 701,000
✓ Calero	$ 361,000	$ 75,000	$ 436,000
Chesbro	$ 304,000	$ 65,000	$ 369,000
Coyote	$ 340,000	$ 75,000	$ 415,000
Guadalupe	$ 302,000	$ 65,000	$ 367,000
✓ Rinconada WTP	$ 60,000	$ 20,000	$ 80,000
✓ Stevens Creek	$ 251,000	$ 50,000	$ 301,000
Uvas	$ 425,000	$ 100,000	$ 525,000
Vasona	$ 40,000	$ 20,000	$ 60,000
Totals	**$ 2,867,500**	**$ 670,000**	**$ 3,537,500**

Note: Details of the instrumentation associated with this estimate are presented in Appendix H of this report.

Table 4-3 Estimated Cost for Dam Instrumentation Project

Projected Cost Element	Estimated Cost
(1) Total Cost of Instrumentation and Installation	$ 3,537,500
(2) Estimated Cost for Final Design and Preparation of Plans and Specifications [20% of (1)]	$ 707,500
(3) Construction Management [10% of (1)]	$ 353,800
(4) Contingency [15% of (1) + (2) + (3)]	$ 689,800
Estimated Total Cost for Dam Instrumentation Plan	**$5,288,600**

There must be a single person who is responsible for the Dam Safety Program. Finally, the most important element in accomplishing all of these efforts is an adequate number of core staff who are experienced in and dedicated to the Dam Safety Program.

Currently, the existing work related to dam safety is being completed by the Water Utility Engineering (WUE) Unit. Staff assigned to dam safety issues includes a full time Senior Civil Engineer, a full time Associate Engineering Geologist, and an Associate Civil Engineer on a one-half time basis. If the Dam Safety Program described in this report is approved by the CEO/GM and the Board, additional staffing will be necessary to complete the anticipated future work. It should be noted that the current staff referenced above does not devote full time to dam safety issues, since they are currently providing engineering and geologic consultation to other major capital District projects, most notably the TWIP project. Fortunately, the requirement for added staff in the WUE for the Dam Safety Program will be staggered over the next six years as the DIP is implemented. The remainder of this section focuses on future staffing requirements, staffing assignments, and the total number of staff associated with the dam safety program.

A variety of tasks must be accomplished in order for the DIP to be successful. These tasks include instrumentation installation, calibration, automation, long-term protection, maintenance, data collection and management, and surveillance report preparation. Generally, these responsibilities can be delegated among individuals with diverse skills and backgrounds, or the same individual can be responsible for various tasks. A summary of the personnel responsibilities associated with maintaining a viable instrumentation program for any dam, or series of dams, is presented in Table 4-4. The overall scope of the work should be supervised by a senior-level geotechnical or instrumentation engineer. Personnel responsibilities are briefly described below.

4.3.1 Senior Geotechnical Engineer/Senior Engineering Geologist

A Senior Level Geotechnical Engineer or Engineering Geologist should be assigned the responsibility of the Dam Safety Program since the major discipline required is overall assessment of the safety of each dam. The senior level implies that the individual has extensive experience and is expert in the performance of slope stability, seepage, consolidation and earthquake analysis. The individual should also have experience in dealing with regulatory authorities and in preparing detailed instrumentation reports for management and regulators. Although it is not absolutely necessary, it would also be a plus for this individual to have a working understanding of automated data acquisition systems, and the MS Access computer program.

4.3.2 Instrumentation and/or Automation Equipment Installation Personnel

Installation should be coordinated by the senior-level geotechnical engineer, since proper installation of instruments and automation equipment requires specific knowledge and requirements. For Lenihan Dam, the District engaged the services of two consultants: one was a registered Engineering Geologist who was responsible for logging all drill holes and installing all instruments; the second was an instrumentation specialist who was responsible for designing the instrumentation automation system and hooking up the instrumentation. This approach was very successful, cost effective, and should be considered for future equipment installations.

Table 4-4 Personnel Responsibilities Associated with the Dam Instrumentation Project

Title	Responsibility
Senior-level geotechnical engineer/Engineering Geologist	Coordinates and supervises the entire Dam Instrumentation Program, including design of instrumentation and automation equipment, selection, procurement, and installation. Provides training to instrumentation personnel and assists in personnel decisions. Should oversee the data interpretation and analysis and provide technical assistance or guidance to other geotechnical engineers or engineering geologists responsible for final data analysis.
Instrumentation and automation equipment maintenance personnel	Maintain instrumentation and automation throughout life of the project. Recalibrate and replace instruments as necessary. Obtain manual readings once per quarter on all automated instruments. This work is currently being completed by an instrumentation consultant, but it is planned that the District will be able to take over maintenance of the entire instrumentation system within about 3 years.
Data collection personnel	Collect data from manually read instrumentation. May also enter data into the computer.
Data management, reporting, and plotting personnel.	Download instrumentation data daily from 11 District dams and evaluate existing conditions for all dams. Using various software programs for data management and evaluation, prepare summary reports for current period for all dams, as appropriate.

4.3.3 Data Collection Personnel

The District still has a limited number of pneumatic and open well piezometers still in use at several dams. These instruments are currently read, usually once per quarter, by Operations Planning and Analysis Unit personnel. Most of the equipment is very old, labor intensive, and will be phased out after new instruments have been installed and several correlative readings have been obtained.

4.3.4 Instrumentation and Automation Maintenance Personnel

The maintenance of dam instrumentation is becoming increasingly more complex due to the implementation of automation at our dams. Currently, for Lenihan Dam, the District has a contract with the original designer of the equipment to provide 24/7 maintenance service to the District because of the numerous alarms that can be triggered at the dam. It is the intent of the District to ultimately provide our own maintenance for all instruments installed as a part of the Dam Safety Program. It should be noted that this function and responsibility will ultimately reside with the Utility Maintenance Unit (control system technician) and Water Utility Engineering Unit (control system engineer). Although these staff provide support to the Dam Safety Program, the staff and budget issues for this will be dealt with separately. Transition of the responsibility from Contractor to District is dependent upon availability of District staff.

4.3.5 Database Management and Reporting Personnel

This position is currently being filled by an Associate Engineering Geologist, who possesses very strong computer graphics skills, and has been directly involved with the data interpretation, plotting, and preparation of surveillance reports for the last several years. As more and more instruments come on line, it is intended to have at least two professionals involved with managing the database. Ideally, this would be an engineering geologist and a civil engineer, both at the Assistant level. The individuals at this level will need training in database management, MS Access, and graphical presentation techniques. Ultimately, both of these individuals will also be trained in maintenance of the instruments to better understand the overall system. It will be imperative for the database management personnel to be competent in handling databases. For example, when considering just the piezometers only, the current instrumentation for Lenihan Dam is generating about 650 bits of data per day, or about 237,000 bits of data per year. If the ultimate instrumentation system that is installed approximates the preliminary design, then the total system will generate slightly in excess of 1,000,000 bits of data per year.

4.3.6 Review and Analysis Personnel

Associate level geotechnical and engineering geologist must be involved in the final review and analysis of all instrumentation data. It will be a goal of the Dam Safety Program to have several personnel at different levels involved so that multiple reviews are being accomplished. The program manager and the technical reviewers will establish data review schedules and formulate reports on a consistent and regular basis as required by DSOD and FERC and, as needed, for management oversight and review.

4.3.7 Dam Maintenance Oversight Personnel

Although it was originally assumed that all new staff additions to the Dam Safety Group would be required as a result of implementing the DIP, as discussed in Section 3.5, routine and planned maintenance at our dams has become a serious issue that could threaten restrictions by DSOD on specific District dams, including Calero, Almaden, Lenihan, and possibly Uvas. As a result, it is recommended that a full-time Associate Civil Engineer position be assigned to the Dam Safety Group as a new position to provide monitoring and oversight to the required environmental issues surrounding dam maintenance issue. The principal projects for this position would include the required seepage mitigation projects at Almaden and Calero Dams, along with the Programmatic EIR for the dam maintenance program. This position has not been added to the staffing requirements discussed in Section 4.4 below, pending discussion with Senior Management.

4.4 Recommended Staffing Requirements for the Dam Safety Program

The staffing required to successfully implement the Dam Safety Program as currently envisioned will include a multi-talented group of professionals specializing in earth dam engineering, automated data acquisition systems, design and maintaining a large database, earth dam emergency response, and the preparation of reports for management and regulators. The projected position classifications requirements for the Dam Safety Program are summarized on Table 4-4.

The current and future staffing requirements for the WUE Unit to successfully implement the Dam Safety Program are presented on Table 4-6. This projection indicates that the staffing requirements will increase from the current (FY 2002) 2.0 person years/year to 4.5 person years/year by FY 2009. The growth of the WUE Unit to handle the Dam Safety Program is directly related to the increasing number of instruments to be installed in our dams, along with the aging nature of our dams. It will be noted that the distribution of future personnel requirements presented on Table 4-6 assumes that no individual will be 100% committed to the Dam Safety Program. It has been assumed for this projection in staffing requirements that additional staff within the WUE Unit will always have some responsibility to other projects within the WUE Unit, such as the Penitencia Creek Landslide and the Calaveras Fault Crossing for example. The responsibility for future staff assignments will of course reside with the WUE Unit Manager.

As shown in Figure 4-1, the current plan is to space out the implementation of the Dam Instrumentation Project over about a 6 year period starting in early-to-mid-2004. If the currently planned schedule is approved and followed, then the number of new instruments added to the monitoring system will approximate the numbers shown on Table 4-7. It is possible that the positions for the Assistant Engineer and the Assistant Engineering Geologist may only be half-time positions, because it is unclear how much addition effort the final automated data acquisition system will require. For the staffing projection shown on Table 4-6, it has been assumed that the personnel requirements to handle the ADAS will roughly parallel the increase in instruments shown on Table 4-7. As shown on Table 4-6, it will be noted that the Senior Engineer and Associate Engineering Geologist positions will, for the foreseeable future, devote at least 1/4 to 1/3 of their time as senior consultants on other major District Projects. It is possible that the assistant-level positions may require full time commitment to the DIP as more and more dams come on line with their automated instrumentation systems.

Table 4-5 Projected Staffing Requirements for the Dam Safety Program

Position Classification	**Responsibilities/Assignments**
Engineering Unit Manager	This position provides overall direction and managerial oversight to the Dam Safety Program staff.
Senior Engineer/Engineering Geologist (Registered Geotechnical Engineer /or Certified Engineering Geologist)	This position will be responsible for day to day management and technical oversight of the Dam Safety Program. The person holding this position will also perform and/or oversee complex geotechnical engineering and engineering geology tasks as required.
Associate Civil Engineer (Registered Civil Engineer)	This position will take the lead in carrying out the civil engineering tasks related to the District's dams, under the direction of the Senior Engineer. Knowledge and experience in earth dam engineering is required to perform all geotechnical engineering analyses for all District dams. This position will also provide redundancy and continuity in the geotechnical side of the Dam Safety Program, in case of the absence of or turnover in the Senior Engineering position, and will also allow for multi-tasking of critical civil/geotechnical engineering tasks.
Associate Engineering Geologist (Registered Engineering Geologist)	This position will take the lead in carrying out the engineering geology tasks related to the District's dams, under the direction of the Senior Engineer. Knowledge and experience in engineering geology as it relates to District dams is required to complement the geotechnical engineering knowledge and experience held by the Senior Engineer.
Assistant Civil Engineer I/II	This position will perform detailed civil/geotechnical engineering tasks under the direction of the Senior Engineer and Associate Engineer, such as stability and seepage analysis. This position will ultimately be responsible for all database management.
Assistant Engineering Geologist I/II	This position will assist the Associate Engineering Geologist in carrying out the engineering geology tasks related to the District's dams. The engineering geology workload required for the Dam Safety Program is more than can be carried out by the Associate Engineering Geologist. This position will also provide redundancy and continuity in the engineering geology side of the Dam Safety Program, in case of the absence of or turnover in the Associate Engineering Geologist (registered) position, and will also provide for multi-tasking of critical engineering geology tasks. This position will also be involved with the monitoring and reviewing the collection of all instrumentation data..
Office Specialist I/II	This (part-time?) position will be responsible for assisting in the preparation of all instrumentation reports and maintaining the files associated with the Dam Safety Program.

Table 4-6 Current and Future Personnel Requirements for Dam Safety Program

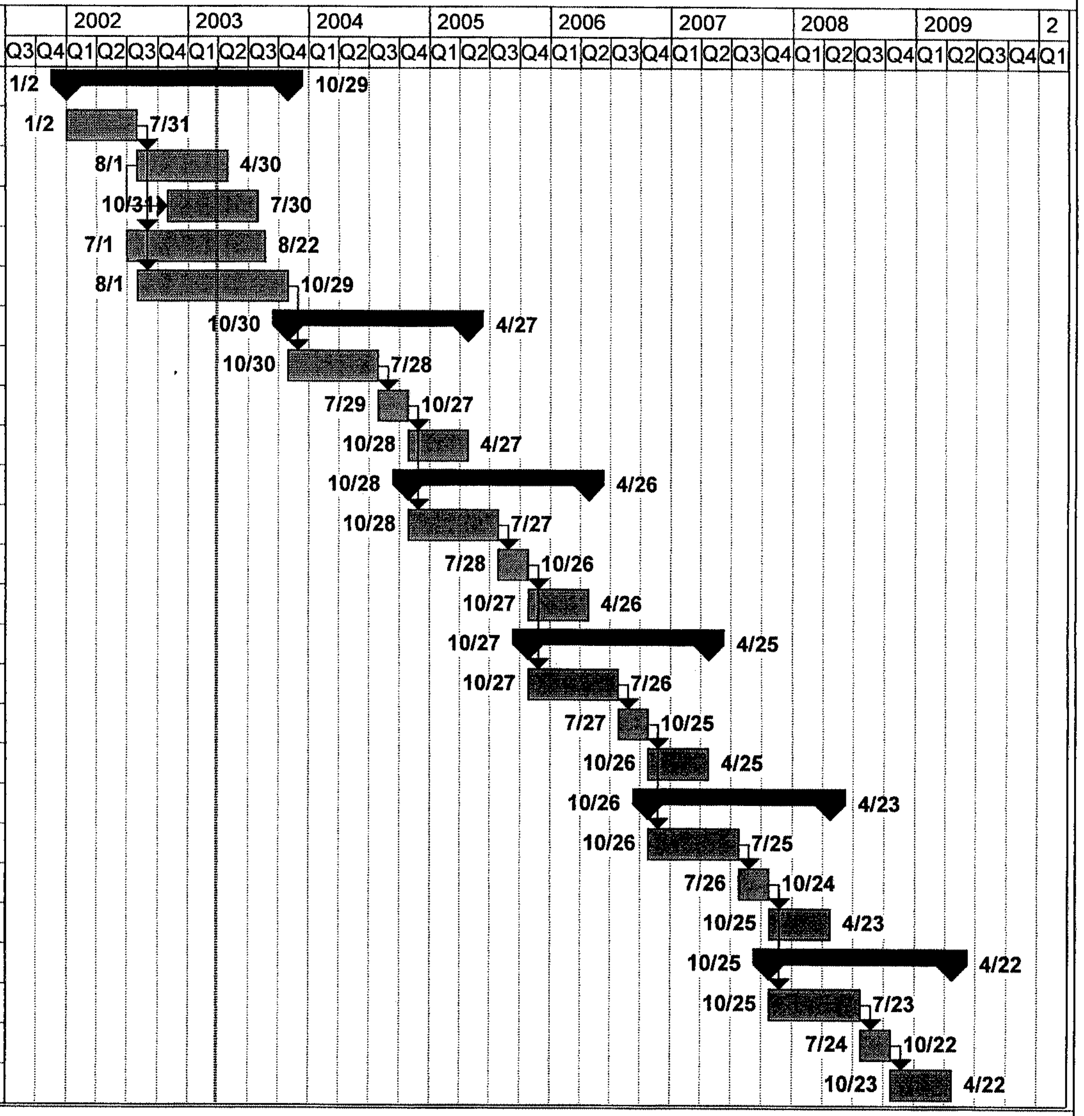
Classification	2002	2003	2004	2005	2006	2007	2008	2009
Senior Engineer	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Associate Engineering Geologist	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Associate Engineer	0.5	0.5	0.5	0.5	0.5	0.75	0.75	0.75
Assistant Engineering Geologist	0.0	0.0	0.5	0.5	0.5	0.5	0.75	0.75
Assistant Engineer	0.0	0.0	0.0	0.5	0.5	0.75	0.75	0.75
Office Specialist	0.0	0.0	0.25	0.5	0.5	0.5	0.5	0.75
Person years/year	2.0	2.0	2.75	3.5	3.5	4.0	4.25	4.5

Table 4-7 Projected Number of Instruments Installed in District Dams

Instrument Type	2002	2003	2004	2005	2006	2007	2008
Electronic Piezometers	54	54	85	121	151	197	214
Electronic Inclinometer	2	2	5	7	9	11	11
Accelerometer	1	1	3	5	5	7	7
Reservoir Level Gauges	1	1	3	5	7	9	11

Figure 4-1 Dam Instrumentation Schedule

ID	🛈	Task Name	Duration
1		**Dam Instrumentation Plan**	**476 days**
2	▦	Detailed instrumentation layout for each dam	151 days
3	▦	Detailed, integrated automation plan (consultant)	39 wks
4		Decision Document/Engineers Report	39 wks
5	▦	CEQA Document (consultant)	60 wks
6		DSOD acceptance of Dam Instrumentation Plan	65 wks
7		**Almaden/Guadalupe Instrumentation**	**390 days**
8		Design	39 wks
9		Bid/Award	13 wks
10		Construction	26 wks
11		**Anderson/Chesbro Instrumentation**	**390 days**
12		Design	39 wks
13		Bid/Award	13 wks
14		Construction	26 wks
15		**Coyote/Calero Instrumentation**	**390 days**
16		Design	39 wks
17		Bid/Award	13 wks
18		Construction	26 wks
19		**Stevens Creek/Uvas Instrumentation**	**390 days**
20		Design	39 wks
21		Bid/Award	13 wks
22		Construction	26 wks
23		**Rinconada/Vasona Instrumentation**	**390 days**
24		Design	39 wks
25		Bid/Award	13 wks
26		Construction	26 wks



Project: dam instrumentation
Date: Tue 3/25/03

Task	▬	Rolled Up Task	▬	Project Summary	▬
Split	⋯⋯⋯	Rolled Up Split	⋯⋯⋯	External Milestone	◆
Progress	▬	Rolled Up Milestone	◇	External Milestone	◆
Milestone	◆	Rolled Up Progress	▬	Deadline	⇩
Summary	▬	External Tasks	▭		

APPENDIX A

STATISTICS FOR DISTRICT DAMS AND RESERVOIRS

APPENDIX A

STATISTICS FOR DISTRICT DAMS and RESERVOIRS

This appendix presents statistics for the District dams and reservoirs. The original dam and reservoir statistics as published in 1936 are presented on Table A-1. Following Table A-1, there are two sheets per dam: 1) a separator sheet containing a recent colored aerial photograph; and 2) a statistics table for each dam. The data are presented in alphabetical order by dam on Tables A-2 through A-11. Statistics on Rinconada Treated Water Reservoir Dam are not included herein since it is not operated as a typical dam, but is used in conjunction with the operation of the Rinconada Water Treatment Plant. The dam statistics are valid through 1998. A comparison of current reservoir data back to Table A-1 reflects the loss in original capacity due to reservoir infilling by silt.

The vertical datum used to reference elevations when the dams were originally constructed was based on the 1929 National Geodetic Vertical Datum (NGVD), which is sometimes referred to as Mean Sea Level. This datum was the basis on which elevations were referenced for the original dam construction and the development of subsequent maps and cross sections. Over the years of reservoir operation, the District has developed extensive storage capacity relationships (i.e., reservoir volume vs. water surface elevation) based on these maps and cross sections. In 1988, a new vertical datum was adopted in the United States that is referred to as the North American Vertical Datum (NAVD). Within the Santa Clara Area, the difference between the two datums is that the 1998 NAVD is about 3 feet higher than the older NGVD. New aerial photos were flown for most of the dams in the 1990s. The resulting new topographic maps for each dam are presented in Chapter 4 dealing with the new Dam Instrumentation Project. Due to the significant database that has developed during the past 60+ years of reservoir operation using the older NGVD, the District continues to record reservoir elevation using the older NGVD datum to determine reservoir volume and other operational planning requirements. For the purpose of this report, however, all elevations are based on the newer NAVD. The dam and reservoir data presented herein were compiled from District files and reports. Special credit and recognition is given to Joe Aguilera in the Operations Planning and Analysis Unit who has maintained and revised (as needed) the reservoir data since the early 1980s.

References:

Santa Clara Valley Water District, History and Statistics, Edited by Gloria Baker, Internal report revised May 1998.

Santa Clara Valley Water District, Dams and Reservoir Specifications and Statistics, Joe R. Aguilera, Management Analyst II, Operations Planning and Analysis, 2003

Table A-1 Original Data on Santa Clara Valley Water District Reservoirs

	Almaden-Calero Unit					
	Almaden	Calero	Coyote	Guadalupe	Stevens Creek	Vasona
Crest length (feet)	460	840	990	650	1,000	1,000
Height above streambed (ft)	105	90	120	129	120	30
Crest width (ft)	20	20	100-160	20	20	20
Base width (ft)	545	495	945	650	620	153
Capacity of Spillway (cfs)	7,000	5,000	33,000	6,000	10,000	12,600
Outlet pipe diameter (ft)	36	36	50	36	50	42
Outlet pipe length (ft)	690	450	900	690	680	20
Reservoir capacity (ac-ft)	2,000	9,500	30,000	3,500	4,000	750
Cubic yards of fill	422,000	722,000	1,200,000	612,000	567,000	62,000
Cubic yards of concrete	3,400	4,000	4,000	5,800	5,700	2,300
Tons of steel	140	166	125	146	200	170
Contractor	McDonald &Bohnett	McDonald &Bohnett	Masco Construction	A. Teichert & Son	McDonald &Bohnett	Carl Swenson Co.
Contract awarded	12-31-34	12-31-34	4-15-35	3-13-35	12-31-34	10-31-34
Final cost	$235,882	$233,404	$674,958	$237,173	$223,118	$115,356

Reference - "Water Conservation in the Santa Clara Valley is Accomplished by Rolled Earthfill Dams" by Fred H. Tibbetts, Water Construction News, V.XI, No. 5, May 1936.

ALAMADEN DAM

Table A-2 Almaden Dam Statistics

Drainage Area (square miles)	12
Year Constructed	1935
Reservoir Capacity (acre-feet)	1,586
Reservoir Surface Area (acres at full pool)	59
Reservoir Length (miles at full pool)	1.1
Dam Type	Rolled Earth Fill
Crest Elevation (feet)	615.0
Dam Height	108
Intake Structural Sill Elevation, feet	540.35
Dam Crest Length, feet	460
Dam Crest Width, feet	20
Dam Base Width, feet	545
Cubic Yards of Fill, cubic yards	422,000
Outlet Pipe Type	Welded Steel Pipe
Outlet Pipe Diameter, inches	36
Outlet Pipe Length, feet	696
Outlet Valves (type and diameter)	2-30" Butterfly
Inlet Valves (type and diameter)	1-42" Sluice
Capacity of Outlet Pipe, cfs	Combined 246, Main 190
Spillway Type	Side Channel
Spillway Capacity, cfs	7,000
Spillway Crest Elevation, feet	606.9
Spillway Weir Length, feet	123
Freeboard, feet	8.1
Contractor	McDonald & Bohnett
Final Construction Cost	$235,882

ANDERSON DAM

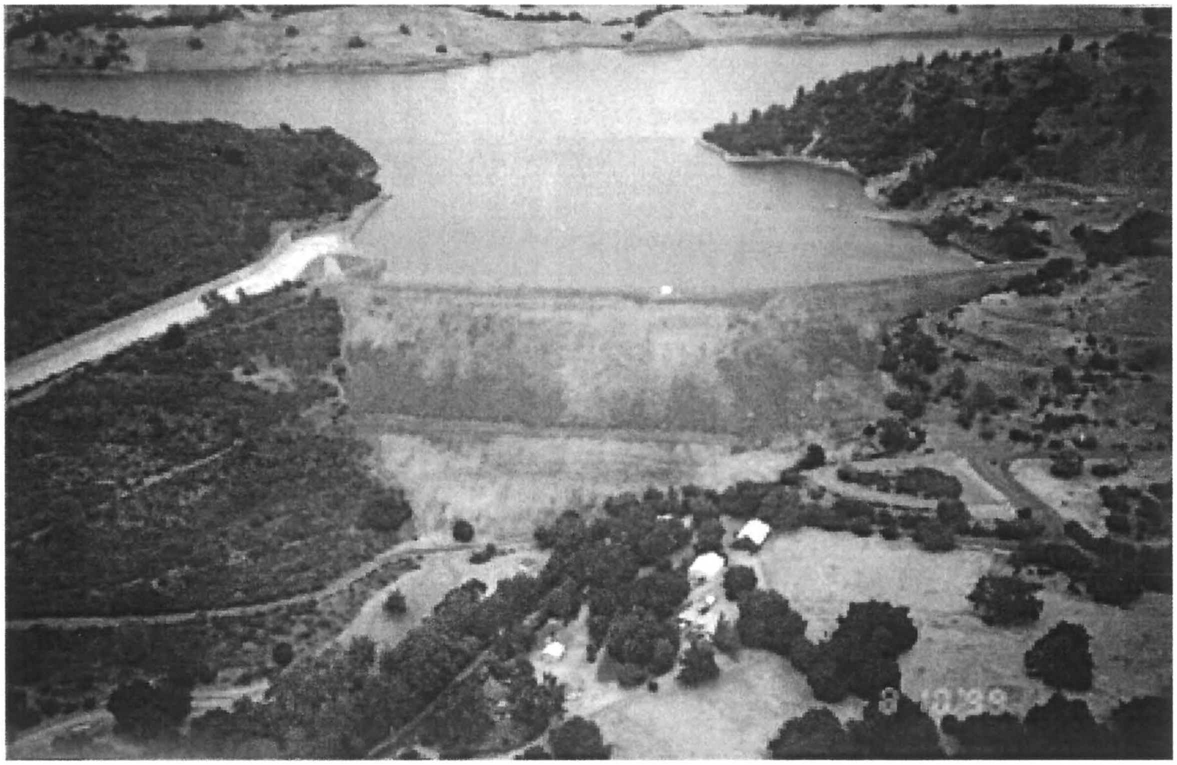


Table A- 3 Anderson Dam Statistics

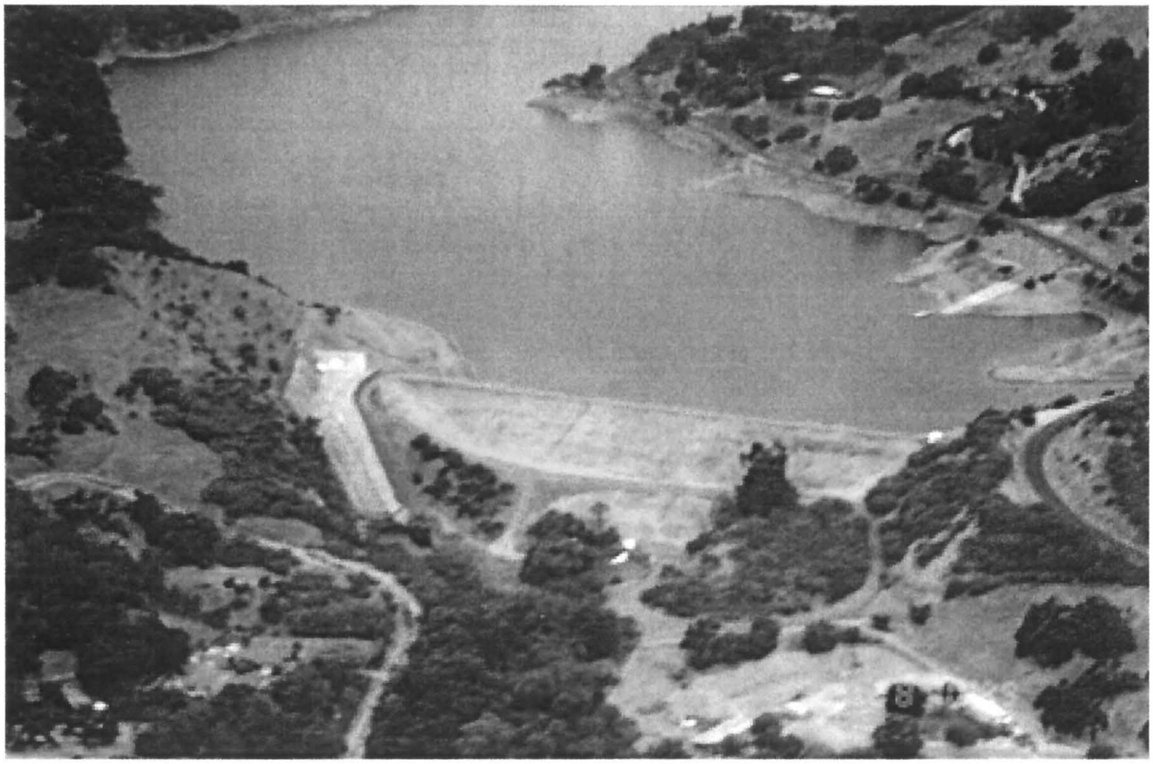
Drainage Area (square miles)	193.4
Year Constructed	1950
Reservoir Capacity (acre-feet)	89,073
Reservoir Surface Area (acres at full pool)	1,245
Reservoir Length (miles at full pool)	7.8
Dam Type	Rolled Earth and Rock Fill
Crest Elevation (feet)	644.5
Dam Height	240
Intake Structural Sill Elevation, feet	560, 525, 485
Dam Crest Length, feet	1,385
Dam Crest Width, feet	40
Dam Base Width, feet	1,100
Cubic Yards of Fill, cubic yards	3,320,000
Outlet Pipe Type	Welded Steel Pipe
Outlet Pipe Diameter, inches	49
Outlet Pipe Length, feet	1,160
Outlet Valves (type and diameter)	12" Polyjet; 48"and 42" Butterfly
Inlet Valves (type and diameter)	Three 60"x84" Sluice
Capacity of Outlet Pipe, cfs	550
Spillway Type	Ogee Chute
Spillway Capacity, cfs	57,400
Spillway Crest Elevation, feet	625.0
Spillway Weir Length, feet	223
Freeboard, feet	15
Contractor	Guy F. Atkinson
Final Construction Cost	$2,225,000

CALERO DAM

Table A-4 Calero Dam Statistics

Drainage Area (square miles)	6.9
Year Constructed	1935
Current Reservoir Capacity (acre-feet)	10,050
Reservoir Surface Area (acres at full pool)	347
Reservoir Length (miles at full pool)	2.2
Dam Type	Earth Dam
Dam Crest Elevation (feet)	490.0
Dam Height	98
Intake Structural Sill Elevation, feet	393.7
Dam Crest Length, feet	840
Dam Crest Width, feet	20
Dam Base Width, feet	495
Cubic Yards of Fill, cubic yards	722,000
Outlet Pipe Type	Welded Steel Pipe
Outlet Pipe Diameter, inches	36
Outlet Pipe Length, feet	481
Outlet Valves (type and diameter)	1-30”
Inlet Valves (type and diameter)	1-42”
Capacity of Outlet Pipe, cfs	185
Spillway Type	Ogee Chute
Spillway Capacity, cfs	5,260
Spillway Crest Elevation, feet	483.5
Spillway Weir Length, feet	82
Freeboard, feet	6.5
Contractor	McDonald & Bohnett
Final Construction Cost	$233,404

CHESBRO DAM



where is the data?

Table A-5 Chesbro Dam Statistics

Drainage Area (square miles)	19.5
Year Constructed	1955
Reservoir Capacity (acre-feet)	8,952
Reservoir Surface Area (acres at full pool)	265
Reservoir Length (miles at full pool)	2.4
Dam Type	Rolled Earth Fill
Crest Elevation (feet)	535
Dam Height	95
Intake Structural Sill Elevation, feet	464.75
Dam Crest Length, feet	690
Dam Crest Width, feet	20
Dam Base Width, feet	428
Cubic Yards of Fill, cubic yards	467,000
Outlet Pipe Type	Welded Steel Pipe
Outlet Pipe Diameter, inches	56
Outlet Pipe Length, feet	480
Outlet Valves (type and diameter)	Butterfly
Inlet Valves (type and diameter)	54 " Butterfly
Capacity of Outlet Pipe, cfs	740 (est)
Spillway Type	Side Channel
Spillway Capacity, cfs	11,000
Spillway Crest Elevation, feet	525
Spillway Weir Length, feet	203
Freeboard, feet	10
Contractor	Blackie & Wood
Final Construction Cost	NA

COYOTE DAM

Table A-6 Coyote Dam Statistics

Drainage Area (square miles)	119
Year Constructed	1935/36
Current Reservoir Capacity (acre-feet)	22,925
Reservoir Surface Area (acres at full pool)	648
Reservoir Length (miles at full pool)	4.8
Dam Type	Earth Dam
Dam Crest Elevation (feet)	804.9
Dam Height	138
Intake Structural Sill Elevation, feet	733.0, 727.8 (2)
Dam Crest Length, feet	970
Dam Crest Width, feet	100
Dam Base Width, feet	945
Cubic Yards of Fill, cubic yards	1,200,000
Outlet Pipe Type	Tunnel (2)
Outlet Pipe Diameter, inches	96(2)
Outlet Pipe Length, feet	702(2)
Outlet Valves (type and diameter)	48"cone – 6" ball(2)
Inlet Valves (type and diameter)	Slide gates
Capacity of Outlet Pipe, cfs	450 (2)
Spillway Type	Ogee section
Spillway Capacity, cfs	33,000
Spillway Crest Elevation, feet	779.9
Spillway Weir Length, feet	110
Freeboard, feet	25
Contractor	Masco Construction
Final Construction Cost	$674,958

Notes:

(1) Datum for elevation is North American Vertical Dam (NAVD), 1988.

(2) A new concrete-lined Outlet Tunnel was constructed in 1990 by Anderson Pacific at a total cost of $4,400,000. The original outlet was grouted closed.

GUADALUPE DAM

Table A-7 Guadalupe Dam Statistics

Drainage Area (square miles)	5.9
Year Constructed	1935
Reservoir Capacity (acre-feet)	3,228
Reservoir Surface Area (acres at full pool)	79
Reservoir Length (miles at full pool)	1.1
Dam Type	Rolled Earth Fill
Crest Elevation (feet)	627
Dam Height	129
Intake Structural Sill Elevation, feet	529.6
Dam Crest Length, feet	650
Dam Crest Width, feet	20
Dam Base Width, feet	650
Cubic Yards of Fill, cubic yards	520,000
Outlet Pipe Type	Welded Steel Pipe
Outlet Pipe Diameter, inches	36
Outlet Pipe Length, feet	720
Outlet Valves (type and diameter)	1-30" Butterfly
Inlet Valves (type and diameter)	1-42" Sluice
Capacity of Outlet Pipe, cfs	235
Spillway Type	Side Channel
Spillway Capacity, cfs	6,000
Spillway Crest Elevation, feet	617.3
Spillway Weir Length, feet	80
Freeboard, feet	9.7
Contractor	Teichert & Sons Inc.
Final Construction Cost	$237,173

LENIHAN DAM

Table A- 8 Lenihan Dam Statistics

Drainage Area (square miles)	36.9
Year Constructed	1952
Reservoir Capacity (acre-feet)	19,044
Reservoir Surface Area (acres at full pool)	475
Reservoir Length (miles at full pool)	2.5
Dam Type	Rolled Earth Fill
Crest Elevation (feet)	666.6
Dam Height	195
Intake Structural Sill Elevation, feet	561.5, 546.0 (gate)
Dam Crest Length, feet	830
Dam Crest Width, feet	40
Dam Base Width, feet	1,370
Cubic Yards of Fill, cubic yards	2,124,000
Outlet Pipe Type	Welded Steel Pipe
Outlet Pipe Diameter, inches	50
Outlet Pipe Length, feet	1,570
Outlet Valves (type and diameter)	None
Inlet Valves (type and diameter)	2-36" Butterfly
Capacity of Outlet Pipe, cfs	410
Spillway Type	Ogee chute
Spillway Capacity, cfs	20,000
Spillway Crest Elevation, feet	649.9
Spillway Weir Length, feet	150
Freeboard, feet	16.7
Contractor	Guy F. Atkinson
Final Construction Cost	$3,350,000*

* Includes costs for both dam and state highway relocation. The cost breakdown is estimated to be about 60% for the dam and 40% for the highway relocation.

STEVENS CREEK DAM

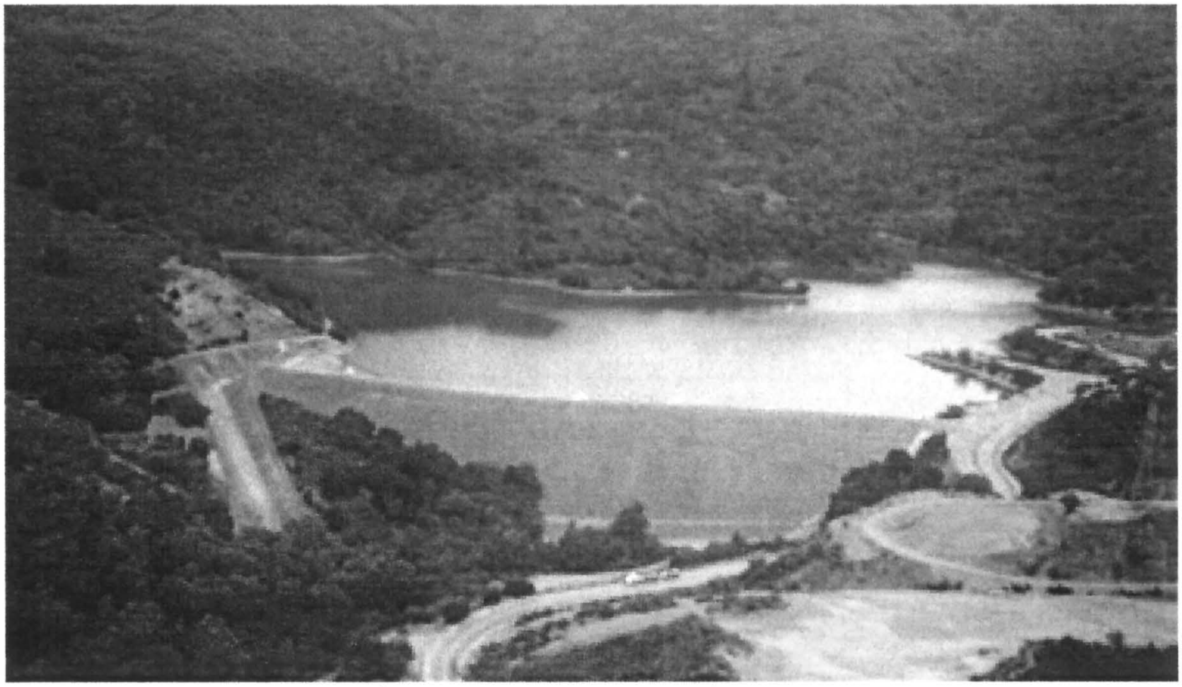


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Table A-9 Stevens Creek Dam Statistics

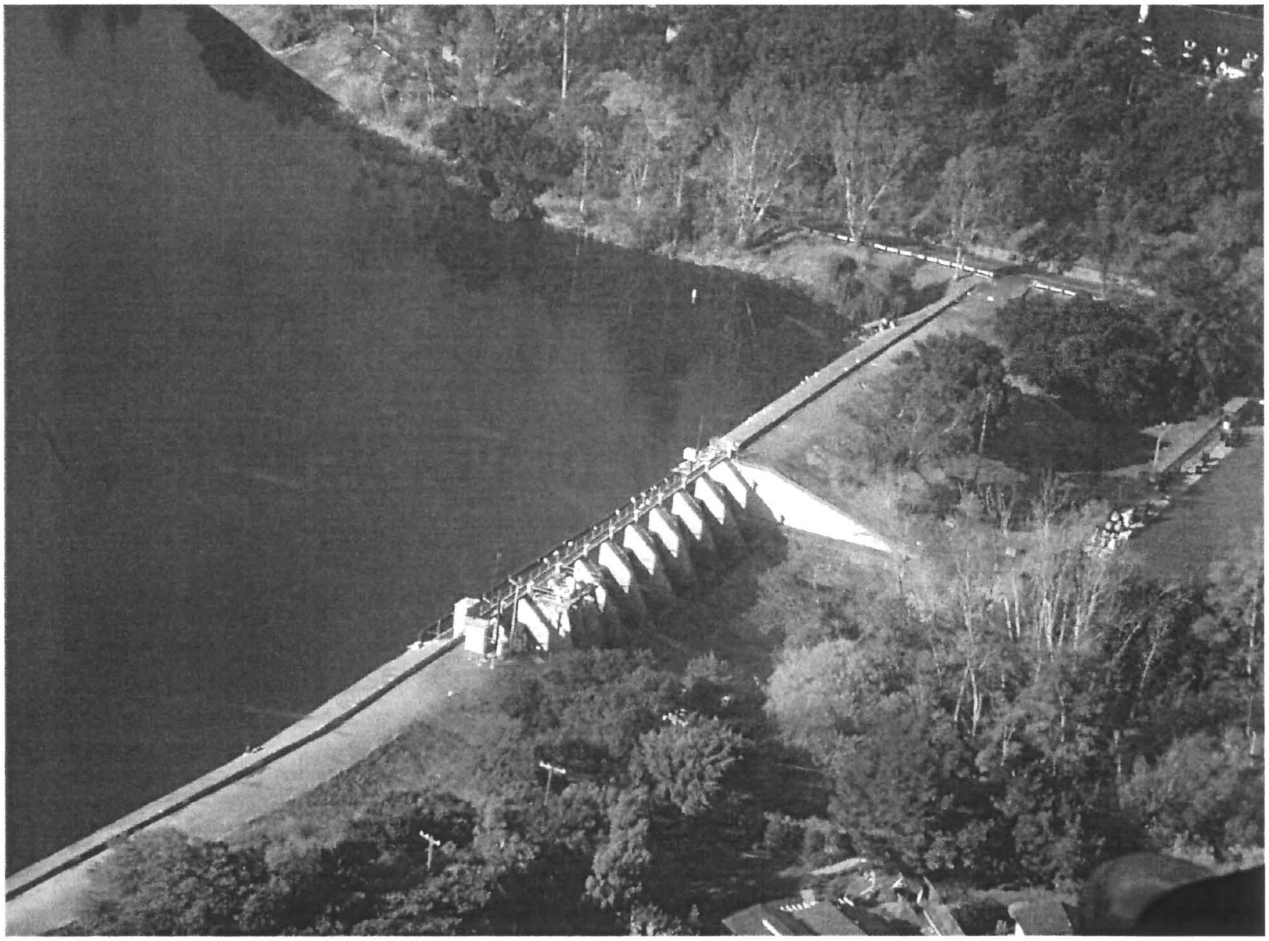
Drainage Area (square miles)	17.5
Year Constructed	1935
Reservoir Capacity (acre-feet)	3,465
Reservoir Surface Area (acres at full pool)	91
Reservoir Length (miles at full pool)	1.1
Dam Type	Rolled Earth Fill
Crest Elevation (feet)	554
Dam Height	129
Intake Structural Sill Elevation, feet	461.0
Dam Crest Length, feet	1,000
Dam Crest Width, feet	20
Dam Base Width, feet	750
Cubic Yards of Fill, cubic yards	770,000
Outlet Pipe Type	Welded Steel Pipe
Outlet Pipe Diameter, inches	50
Outlet Pipe Length, feet	890
Outlet Valves (type and diameter)	2-30" Butterfly
Inlet Valves (type and diameter)	2-42" Sluice
Capacity of Outlet Pipe, cfs	410
Spillway Type	Side Channel
Spillway Capacity, cfs	15,715
Spillway Crest Elevation, feet	534.9
Spillway Weir Length, feet	172
Freeboard, feet	19.2
Contractor	F. W. Moore
Final Construction Cost	$223,118

UVAS DAM

Table A-10 Uvas Dam Statistics

Drainage Area (square miles)	32
Year Constructed	1957
Reservoir Capacity (acre-feet)	9,935
Reservoir Surface Area (acres at full pool)	286
Reservoir Length (miles at full pool)	2.2
Dam Type	Rolled Earth and Rock Fill
Crest Elevation (feet)	500
Dam Height	105
Intake Structural Sill Elevation, feet	410.0
Dam Crest Length, feet	1,100
Dam Crest Width, feet	20
Dam Base Width, feet	600
Cubic Yards of Fill, cubic yards	800,000
Outlet Pipe Type	Welded Steel Pipe
Outlet Pipe Diameter, inches	36
Outlet Pipe Length, feet	850
Outlet Valves (type and diameter)	1-30" Butterfly and 1-20" Gate
Inlet Valves (type and diameter)	1-42" Sluice
Capacity of Outlet Pipe, cfs	165
Spillway Type	Ogee Chute
Spillway Capacity, cfs	N/A
Spillway Crest Elevation, feet	487.5
Spillway Weir Length, feet	N/A
Freeboard, feet	12.5
Contractor	Blackie & Wood
Final Construction Cost	N/A

VASONA PERCOLATION DAM



Data?

Table A-11 Vasona Dam Statistics

Drainage Area (square miles)	43.9
Year Constructed	1935
Reservoir Capacity (acre-feet)	400
Reservoir Surface Area (acres at full pool)	57
Reservoir Length (miles at full pool)	0.8
Dam Type	Rolled Earth and Concrete Buttress
Crest Elevation (feet)	305
Dam Height	30
Intake Structural Sill Elevation, feet	275
Dam Crest Length, feet	1,000
Dam Crest Width, feet	20
Dam Base Width, feet	153
Cubic Yards of Fill, cubic yards	70,000
Outlet Pipe Type	N/A
Outlet Pipe Diameter, inches	42
Outlet Pipe Length, feet	20
Outlet Valves (type and diameter)	1-42" Slide Gate and 2-13' x 10' radial gates
Inlet Valves (type and diameter)	None
Capacity of Outlet Pipe, cfs	125
Spillway Type	Concrete buttress
Spillway Capacity, cfs	12,600
Spillway Crest Elevation, feet	294.8
Spillway Weir Length, feet	N/A
Freeboard, feet	10
Contractor	Carl Swenson Co.
Final Construction Cost	$115,356

APPENDIX B

CULTURAL AND CONSTRUCTION HISTORY OF DISTRICT DAMS

APPENDIX B

CULTURAL AND CONSTRUCTION HISTORY OF DISTRICT DAMS

1.0 Introduction

In the introduction to this report, it was noted that the cultural history of the land currently owned and maintained by the Santa Clara Valley Water District dates back several centuries to its Spanish heritage. Some of this cultural history is summarized in this appendix. The information on cultural history comes from a series of pamphlets published by the Santa Clara County Parks Department, and augmented by information in District files. The discussion on construction history was gleaned from District files. The topics and length of discussion covered under construction history differ for each dam depending on the various issues that dominated the contractors or engineers during construction. Finally, it will be noted that Chapter 1 presents location maps for District dams and reservoirs and Appendix A presents recent aerial photographs and current statistics for each dam and reservoir. All distances and elevations referenced in this appendix are in feet.

2.0 Almaden Dam

2.1 Cultural History

About 10 million years ago, hydrothermal activity, caused by tectonic plate movement changed some of the park's serpentine rock into silica carbonate, depositing high grade mercury sulphide (HgS), the principal ore mineral of mercury known as Cinnabar. The Native America population of Ohlone Indians used the red ore, which they called "mohetka," as a rich red pigment for facial and body painting. This red ore would later make the park site the first and richest mercury mine in California's history, and the largest mercury mine operation in the U.S.

Andres Castillero, a Mexican cavalry officer, arrived in the San Jose area in 1845. The Ohlone Indians brought Castillero to the source of the red ore, after which he filed a claim and was granted title to the mineral deposit by the Mexican government. The mine site was within the boundaries of Rancho San Vicente, which had been granted to Jose Reyes Berryessa several years earlier. Castillero set up camp near the Los Alamitos Creek, but within a few months returned to Mexico to seek financing. In 1846, war broke out between the U.S. and Mexico and Castillero never returned.

In late 1846, the firm of Barron, Forbes Company acquired the shares of Castillero, securing a majority ownership in the mineral deposits. In the interim, the minority ownership had taken charge of the operation and challenged the legality of the Barron, Forbes Company, which was refuted based on the boundaries of the mining grant. The Barron, Forbes Company evicted the minority ownership and began developing a settlement later known as the "Hacienda de Beneficio," or reduction works. This area would later be known as the "Hacienda" and the mines were named the New Almaden Mines after the world renowned Almaden Mines located about 150 miles southwest of Madrid Spain. The New Almaden mercury mine was in part responsible for the success of the California gold and silver mines. By 1854, thirteen furnaces were in continuous operation and in 1856, the New Tunnel (Day Tunnel) was constructed under the direction of the superintendent and surveyor, Sherman Day. The workers built houses on the ridge above Deep Gulch which would later be named Spanishtown. Spanishtown, with mostly a Mexican population, was the largest of the mining settlements. The Hacienda, known today as New Almaden, included the Casa Grande, built by Francis Meyers in 1854 and was the official residence of the mine manager. Today, the Casa Grande, part of the Almaden Quicksilver County Park, is home to the Quicksilver Mining Museum.

Once California became a state in 1850, all Mexican land grantees were required to re-file their claims with the U.S. Government. The Barron, Forbes Company filed a claim in 1852 to establish the legality of the Castillero claim. The documents were found to be insufficient and a legal battle ensued which would carry on for twelve years, ending with a U.S. Supreme Court ruling against the Barron, Forbes Company. Following the ruling in 1862, President Lincoln signed an order to evict the Barron, Forbes Company. Under public pressure, Lincoln withdrew the eviction order and in 1864 the Barron, Forbes Company relinquished ownership of the 8,500 acres for $1.75 million to the Quicksilver Mining Company. Samuel Butterworth, president of the Quicksilver Mining Company, assumed management of the mines in 1864. He was determined to increase the production of the mines and improve the general welfare of the mining community.

During Butterworth's tenure, a company store and a schoolhouse were built on Mine Hill, a tramway system to transport ore was constructed, and rules were established to regulate working and living conditions. By 1865 there were over seven hundred buildings, mostly dwellings, associated with the mine, with 1,800 people living on Mine Hill. The 1860s brought an influx of Cornish miners who established the settlement of Englishtown on Mine Hill. The town included a schoolhouse, boarding house, a community center, and a Methodist Episcopal Church.

In 1870, Butterworth resigned and was replaced by James B. Randol. In 1871, Randol initiated the construction of a new shaft for deeper exploration known as the Randol shaft, which would operate through 1896 and would become the richest of the mines, producing about $10 million in quicksilver. A new entry for the shaft, the Santa Isabel opening, was created in 1877 and accommodated three hoisting compartments reaching a depth of 2,300 feet. By 1874, five new furnaces had been installed, processing 145 tons of ore per day. The Buena Vista Shaft was developed with massive granite block foundations to hold a large pump engine. Under Randol, the mines reached their peak of development. Randol also improved the health and welfare system for the miners and their families in 1871 by establishing a Miner's Fund health plan to finance a resident camp doctor. At Randol's retirement in 1892, Robert Bulmore was appointed Mine Manager, a position he held until 1899. With the closing of the mine operations in 1912, Englishtown and Spanishtown became deserted. Upon the decline of the Mine Hill operations, attention was directed to Senador Mine (Senator in English) where a new shaft was constructed in 1909. By 1912, operations were limited to working the mine and furnace yard tailings, and the Hacienda furnace yard was stripped. In 1915, mining facilities at the Senador Mine were expanded to include a reduction plant, but by 1927 exploration had failed and the New Almaden Corporation was declared bankrupt. Parts of the furnaces still remain at Senador Mine.

In the 1930s the Civilian Conservation Corps (CCC), and element of the Works Progress Administration (WPA), was established by the Federal Government. Six CCC companies served consecutive years from 1933 through 1939 on Mine Hill. The six companies occupied Englishtown, building fire roads and lookout towers while serving as firefighters. A monument to the CCC stands on Mine Hill.

From 1928 to 1972, mining operations continued under small operators, including the New Idria Mining Corporation. Operations commenced again in 1940 under the New Almaden Corporation and continued through World War II. During this time, a 100-ton rotary furnace was constructed below Spanishtown's Big Rock. By 1970, the decline in the price of mercury and the realization of mercury's environmental toxicity caused the mines to close. All operations ceased in 1976.

In 1973, the County of Santa Clara acquired all of the New Idria Mining Company property outside the Mine Hill area amounting to 2,455 acres. In 1975, the County acquired the balance of the property (1,115 acres) from New Idria Mining Company, and opened the Almaden Quicksilver County Park to the public. An additional 408 acres have been added since 1976.

With over a century of quicksilver production, the New Almaden mines have made their mark on the growth of California. All that remains today of this once vibrant area are a few lonely landmarks to memorialize the bonanza days of the New Almaden mines. One lone chimney stands above the reduction works site at the Hacienda entrance and the Powder House, once used to store the black powder used for blasting, stands alone on the April trail. Also remaining is the mercury in the watershed, which remains a significant solution problem for the County.

2.2 Construction History

Almaden Dam, one of the six original District Dams, was built in 1935 across Alamitos Creek. Except for Vasona Lake, which now acts as a holding area for the recharge ponds in the Campbell area adjacent to Los Gatos Creek, Almaden reservoir is the smallest of the District reservoirs with a current capacity of 1,553 acre-ft. and a watershed area of about 12 square miles. Almaden Dam was designed and constructed as a zoned earthfill dam with an impervious zone in the upstream half and a pervious zone in the downstream half. Zoning within the dam was achieved by using relatively finer-grained materials in the impervious zone and coarser-grained materials in the pervious zone. Since the zoning was achieved by selective routing of construction hauling equipment within a single borrow area for the embankment fills there does not appear to be a significant distinction between the soil properties of the impervious zone and pervious zone.

In 1935, engineering geologist Chester Marliave logged foundation earth materials encountered during dam construction excavation for the cut-off trench and outlet pipe alignments and provided logs in a 1936 report. Marliave characterizes the vicinity of the site as being underlain by Franciscan Complex "*sediments*" of "*clay shales, cherts, and sandstones into which have been intruded basic igneous rocks which are now metamorphosed into serpentine and its derivatives, the silica carbonates.*" Specifically, Marliave logged Franciscan Complex clay shale in the vicinity of the right abutment, and alternating masses of schist, black shale, black shale schist, and minor sandstone along the outlet pipe alignment. A large mass of serpentinized ultramafic rocks (serpentinite) was logged in the left abutment and thin zones of serpentinite were logged along the outlet pipe. Silica carbonate rock was encountered in the left abutment during excavation of the cut-off trench. Serpentinite masses are believed to have formed as the result of thrusting of oceanic crust into Franciscan Complex rocks during abduction of these rocks onto the North American Continent during the Jurassic. Silica carbonate was formed by hydrothermal alteration of serpentinite, probably during the Miocene.

Geologists would now classify Marliave's "black shale/black shale schist" and serpentinite masses as shear zone or melange based on his descriptions of serpentinite that it is "*badly sheared*" "*very soft and mushy*" and of shale that is "*badly crushed*" and "*gougy*." Gouge is a geologic term for sheared materials along faults or shear planes beneath landslides. Franciscan Complex melange typically consists of relatively hard coherent blocks of various rock types surrounded by highly sheared shaley matrix. Recent mapping by U.S. Geological Survey geologists McLaughlin and Helley (2001) indicate that the dam and reservoir are surrounded by Franciscan Complex melange.

The as-built drawings of Almaden Dam and foundation show construction changes in alignment of the cut-off trench and the dam crest in the left abutment due to the presence of silica carbonate rock. These changes resulted in construction of a "dogleg" or bend in alignment of the dam crest. The presence of a fillet of fill at the downstream left groin may also be related to final construction changes due to the presence of silica carbonate rock and concerns of the original builders about seepage through the left abutment.

A picture on page 28 of Marliave's report (1936) dated 1/30/36, shows the fillet of fill. The caption below the picture states: *"Looking along the axis of the completed dam toward the left abutment. Picture shows the impounding of the first flood waters since the dam was constructed. The viewpoint is at the top of the slope of the spillway cut which is just out of sight in the foreground."*

It should be noted that the fillet of fill was never included on the "As-Constructed" map. Due to lack of information on the additional fill, we do not know the type earth materials contained in the fillet or how it was placed. To obtain additional information on the fillet of fill, we plan to investigate the fill during our upcoming Dam Instrumentation Project.

3.0 Leroy Anderson Dam

3.1 Cultural History

For centuries, the Tamien and Matalan Tribes of the Ohlone Indians lived along what is now the Coyote Creek Parkway. The temperate climate, abundant wildlife and plant food sources attracted these Native Americans to this site. In March of 1776, Juan Bautista de Anza and his party of explorers crossed the river on their way to San Francisco. They named the river Arroyo Del Coyote, most likely after the coyotes they saw during their journey.

The Coyote Creek Parkway also includes an area at the terminus of Burnett Avenue in Morgan Hill commonly called the "Burnett Area." This portion of the park is part of the Spanish land grant of Rancho De Laguna Seca. The area has been used over the years for cattle grazing, chicken ranching, and orchard farming.

Anderson Reservoir, named for Leroy Anderson, is the largest lake in Santa Clara County. Anderson was a devoted conservationist and one of the original leaders of the Santa Clara Valley Water Conservation Committee, the predecessor to the current District. Construction of the lake and dam was funded from a $3 million Bond Act approved by voters in 1949. Anderson Reservoir now serves as part of the Santa Clara Valley Water District's linked system of reservoirs for groundwater recharge and as a source of water for the District's water treatment plants.

At the bottom of what is now Anderson Reservoir is land once farmed by the Cochrane family. In 1869, John Cochrane purchased part of the Spanish land grant Ojo de Aqua de la Coche. Cochrane came from New Hampshire, and had sailed around Cape Horn to reach California. He worked the ranch as a dairy using Swiss farmhands. At its peak, the ranch encompassed 5,500 acres in the Mount Hamilton foothills.

Cochrane and his wife, Aphelia Farmington, had five children. After Cochrane's death in 1899, his wife took over the ranch operation. She began raising beef cattle after the 1906 earthquake destroyed the stone dairy barn. In 1914, Aphelia built a new ranch house near the original adobe house. Over the years, she become a prominent citizen of Morgan Hill, and died in 1949 at the age of 103. The following year, her estate was awarded $155,000 for 500 acres along the Coyote Creek where Anderson Dam and Reservoir were constructed. The new house and some other buildings were moved to the lake's southeastern shoreline where they remain today.

Gladys Jackson, one of the Cochrane granddaughters, and her foster sister Ruth Lowe, lived at the ranch until it was badly damaged in the 1984 Morgan Hill earthquake. Soon after, Gladys died at the age of 88. Ruth died in 1987 at age 91. Gladys Jackson's will deeded the ranch to the California Pioneers of Santa Clara County. The ranch house was recently restored and is now part of Anderson Lake County Park.

3.2 Construction History

Leroy Anderson Dam is a zoned earth-rock embankment with a maximum height of about 240 feet as measured from the streambed to the crest. At the time of its completion in 1950, it was the highest embankment dam in northern California. It has a crest length of about 1,400 feet, a crest width of 26 feet (except in the vicinity of the old control building, where it is 43 feet) at a crest elevation of about 645 feet (without camber), and uniform upstream and downstream slopes of about 2.5:1 (H:V). The reservoir has a capacity of 89,073 acre-feet at a spillway crest Elevation of 625 feet, and a drainage area of 193 square miles. Reservoir storage is released primarily for groundwater recharge and to District water treatment plants to produce potable water. Secondary benefits include flood control and recreation.

The dam and spillway were constructed between March 1950 and November 1950 (the outlet works were constructed in late 1949 and early 1950). The dam and appurtenant structures were designed and plans and specifications prepared by the (then) Santa Clara Valley Water Conservation District (SCVWCD). Guy F. Atkinson Company of South San Francisco constructed the embankment and construction inspection and testing were provided by the SCVWCD. The State of California Division of Safety of Dams (DSOD) made frequent inspections and retained the services of Mr. J. L. Savage, Consulting Engineer, and Dr. G. D. Louderback, Consulting Geologist, to review construction progress and help resolve special problems. The DSOD also provided a full-time resident inspector during most of the construction. Problems during construction were chiefly associated with the condition and quality of the foundation rock. In the right abutment, the issue was resolved by extensive stripping and excavation of a deep cutoff trench. In the left abutment area, the foundation issue was resolved primarily by shifting the dam axis 150 feet upstream to secure a more impervious foundation beneath the core. This movement in the alignment resulted in the crest having an angle point in the left half of the dam.

3.2.1 Geologic Setting & Key Issues

Anderson Dam and its appurtenant structures are underlain by Franciscan Complex bedrock. Sandstone (graywacke), shale, and greenstone, comprise much of the dam foundation. The sandstone and greenstone occur as competent, variously-sized blocks in a matrix of crushed (sheared) shale. Serpentinite comprises the dam foundation in the upstream channel section and the upper right abutment, and is exposed in the spillway approach channel and along most of the right side of the lined section of the spillway. The serpentinite occurs as boulder, cobble and smaller-size pieces in a matrix of crushed serpentinite. The sandstone, shale, greenstone suite is in fault contact with the serpentinite in the upstream channel section of the dam foundation (Wahler Associates, 1984). Terrace deposits occur widely at the site. They are exposed along the spillway approach channel and along the right side of the spillway. The terrace deposit is a fluvial/interfluvial deposit, considered to be Pleistocene-age (Ibid.) and is possibly correlative with the Santa Clara Formation. The Coyote Creek fault is evident in the boat ramp cut upstream of the left abutment and is mapped on the right side of the spillway and spillway approach channel (Ibid.). Traces of the Coyote Creek fault cross the upstream dam foundation including one trace which delineates the contact between the serpentinite and the sandstone, shale, greenstone suite. Serpentinite is displaced over a basal unit of terrace deposits along a trace of the fault in an area of over-steepened slope along the approach channel. The fault is truncated by terrace deposits and the latest age of faulting of 100,000 years was ascribed to this trace (Shlemon, 1983).

The principal geotechnical issues that confronted the design of the dam were related to:

(1) its unprecedented height of 240 feet and the somewhat questionable nature of the bearing capacity of the faulted bedrock in the foundation;
(2) the questionable nature and rock quality of the right abutment bedrock rock from which the shell materials were to be derived, and;
(3) the proximity of the Coyote Creek and Calaveras fault systems.

3.2.2 Dam Construction

Anderson Dam is a central core rock fill dam. The slopes of the upstream and downstream rockfill are inclined at 2.5(H):1(V), and the symmetrical impervious core slopes at 0.75(H):1(V). The impervious core is composed of two zones: namely, an upstream Zone 2 consisting of approximately 382,000 cubic yards, and a downstream Zone 3 consisting of 776,000 cubic yards. The combined top width of the core is 40 feet and the bottom width is about 375 feet at the maximum section. The maximum stripping depth beneath the core was about 32 feet.

The upstream Zone 2 material was derived from required excavations in the foundation, spillway, and spillway outlet channel. The finer grained downstream Zone 3 material was derived from a borrow source located within the reservoir. The average gradation of the Zone 3 material has a maximum grain size of 1-1/2 inch, 85% finer than the No. 4 sieve (4.75 mm), 53% finer than the No. 200 sieve (0.075 mm), and 20% clay fines (0.002 mm). This corresponds to about 15% gravels, 32% sands, and 53% fines. The fine material is a medium-plastic clay with an average Liquid Limit of 42%. The natural water content of the core ranged from 4% below (unsaturated zone) to 3% above (saturated zone) the Plastic Limit of the clay.

The impervious core is quite strong and impervious. A review of construction records indicates that the material was compacted in 4-6 inch lifts with a minimum of 10 passes of a heavy tamping roller which produced a minimum relative compaction of between 95% to 98%, based on a compactive energy of 36,000 ft-lb./cu.ft. Laboratory tests performed on undisturbed samples indicate that the shear strength of the core consists of a friction angle of 28° and a cohesion of 890 psf. The coefficient of permeability was measured at 2.5×10^{-8} cm/sec (W. A. Wahler & Associates, 1976).

The upstream (Zone 1) and the downstream (Zone 4) shells of Anderson Dam consist of rockfill material derived from two different quarries located very near the dam site. Zone 1 contains approximately 970,000 cubic yards of material principally derived from a quarry in Franciscan metavolcanic rock in the left abutment above el. 680. The initial quarry source for Zone 1 was in silica-carbonate rock on the right abutment upstream of the dam axis. Records indicate, however, that only two 15-20-foot high lifts of this rock were placed (up to el. 440) before the quarry was abandoned in favor of the left abutment site. Zone 4 contains approximately 1,200,000 cubic yards of rockfill derived from metavolcanic rock quarried north of Coyote Creek at the lower end of the spillway outflow channel. Both Zone 1 and Zone 4 were placed by end-dumping of trucks onto lifts of variable thickness (10 to 25 feet). Approximately two (2) cubic yards of water per cubic yard of material was applied to the rockfill through high-pressure hoses (monitors) as the material was being dumped from the trucks. After spreading and smoothing by a dozer, each lift was proof rolled with two passes of the same tamping roller used in the compaction of the core. Anderson Dam represents one of the last rockfill dams (early 1950s) built in the United States using such a high lift thickness and large quantities of water. Newer construction techniques for rockfill dams incorporate 4-8 ft thick lifts that are compacted with heavy, smooth drum vibratory rollers.

3.2.3 Spillway

The spillway was modified in 1987 and 1988 to pass the Probable Maximum Flood which, according to design records, has a peak inflow of 60,224 cfs and outflow of 57,400 cfs. The spillway is ungated with an ogee crest section and a concrete lined chute terminating in a dentated flip bucket. At the end of the chute, the spillway drops off vertically into a discharge channel, which is excavated in rock, and has a riprap protected bottom channel immediately adjacent to the end of the spillway, and riprap protected sideslopes up to the base of the spillway floor (Elevation 535) for a distance of about 200 feet downstream. In order to provide sufficient head to pass the PMF over the modified spillway, the crest of the dam was raised about five feet. Although the spillway lip remained essentially unchanged at el. 625.1, the raising of the crest increased the freeboard from about 14.9 feet to 19.9 feet.

3.2.4 Intake and Outlet Works

The original intake structure was located at the upstream toe of the dam. It connected to the outlet pipe which consisted of a single 1,160 foot long, 49 1/8-inch ID steel pipe, having a wall thickness of 5/16 inch, in an 18-inch-thick reinforced concrete encasement. It passes beneath the center section of the embankment. Discharge is into a 150-foot-long reinforced concrete channel. The invert of the downstream (discharge) exit is Elevation 404.6. The upstream inlet was modified in 1988-1989 due to reservoir siltation. The new multi-port inlet structure was considered necessary because analysis showed that if the original inlet had been raised, the structure would have been structurally vulnerable to strong seismic shaking. The decision to construct a new multi-port intake in the upstream left abutment was based on the following reasons: 1) a structural safety concern, related to a higher perceived seismic requirement, regarding the need to raise the original intake structure even higher than the 1960 raise; 2) to allow flexibility withdrawing water from the reservoir at selected levels; 3) to provide control for the water temperature and water quality; and 4) to design an intake system that would not be hampered by future reservoir siltation concerns.

The multi-port intake allows water to be withdrawn from three different elevations; 485, 525, and 560 feet. The intakes are controlled independently by hydraulically operated, horizontally-mounted sluice gates. Each gate has a 60-inch by 84-inch opening. Gate operation is initiated manually from a reinforced concrete block Control Building. Buried hydraulic lines lead from the Control Building to the gate operators. The Control Building is located on the south (left) abutment above the intake structure. An electric motor is the primary power source for the hydraulic system. A gasoline engine is provided as back-up to be used in the case of failure of the electrical system. Both the auxiliary power sources and associated equipment are located in the Control Building.

The new multi-port intake has been very cost effective. Prior to the new facility, the outlet water obviously came from the lowest (and coldest) point in the reservoir where the total dissolved solids were high. With the new facility, the District can obtain water at different elevations based on optimum reservoir water temperature, and lower total dissolved solids, thereby limiting the need for more expensive water treatment. The reduction in related water treatment costs essentially paid for the new multi-port facilities within about three years of its installation.

4.0 Calero Dam

4.1 Cultural History

The valley that underlies Calero Reservoir and its surrounding area offered a rich environment to the Matalan Ohlone Indians. Four pre-contact archeological sites are known to exist within the Calero County park. During the Spanish/Mexican colonization of Santa Clara County, the present day Calero Park included parts of the large Rancho San Vincente land grant and portions of the Pueblo of San Jose.

The Bailey Fellows house is a historic "Italianate" Victorian house located at the south end of Calero Reservoir. This house, built by Boargenes Bailey around 1870, was later owned by Judicial District Court Judge Edward Fellows. The house is now part of Calero Ranch Stables, and is not currently open to the public. The master plan for Calero County Park proposes to renovate the house and create a visitor center, museum and special event area.

Boargenes R. Bailey was born in Tennessee in 1827. Boargenes and his brother, Doctor Bowling Bailey, arrived in California by wagon in 1850. For six months, the brothers prospected in the Sierra foothills and eventually settled in Mountain View. In 1855, Boargenes married Ellen Sparks, who arrived in California with the Lassen Party in 1848. In 1867, the Baileys purchased 873 acres of the City of San Jose Pueblo Lands. The Baileys engaged in stock raising, orcharding and farming in what was then known as Calero Valley. The Bailey family included nine children. Boargenes was a trustee of the Llagas School District and donated land for the school in 1875. B.R. Bailey died in 1883. Mrs. Bailey remained on the property until 1903. The property was sold to the Newman Brothers in 1905 and was known as the Newman Ranch. In 1935, the property was sold to the Santa Clara Valley Water Conservation District for the proposed Calero Reservoir.

During construction of the reservoir, a levee was built around the house, ranch and orchards and is now referred to as the Fellows Dike. Between 1936 and 1966, Edward and Triscilla Fellows owned and operated the "Calero Ranch". Edward Fellows was born in Santa Clara County in 1900 to immigrant parents from the Azores. His wife, Triscilla Nicora Fellows, was born in San Jose in 1898. Edward Fellows completed his undergraduate work and Law School at Santa Clara University in 1923 and began his law practice in San Jose in 1924. In 1925, Ed Fellows married Triscilla Nicora of San Jose. In 1938, the Fellows purchased the 2,000-acre "Calero Rancho" and operated the cattle ranch and farm. The house was remodeled by Triscilla Fellows in the 1940s. The Fellows hosted many gatherings at the ranch, including the Santa Clara County Bar Association's annual barbeque. In 1956, Edward Fellows was sworn in as Judicial District Court Judge for Morgan Hill. Judge Fellows died in a tragic train/auto crash in 1965. The house and stables were sold to the Santa Clara Valley Water District in 1967. The house is registered with the Historical Heritage Commission of Santa Clara County and currently serves as the office for Calero Ranch Stables.

In 1968, the County began leasing the reservoir lands from the Water District and developed facilities at the reservoir for water-oriented recreation. The lands known as the back country were acquired by the Parks Department in the late 1960s and 1970s primarily from the owners of the Fellows ranch, which is also known as the Calero Ranch Trust.

4.2 Construction History

Calero Dam consists of two principal dams, the 97-ft high and 840-ft long main dam located across Calero Creek, and the Auxiliary Dam. The Auxiliary Dam, also referred to as a saddle dam, is located about 3,500 feet east of the main dam, is about 40 feet high, and has a crest length of about 500 feet. The saddle dam, constructed in 1935 at the same time as the main dam, was necessary to contain the reservoir from flowing into a small drainage adjacent to the new alignment of McKean Road. Both Calero dams were designed and constructed as zoned earthfill dams with an impervious zone in the upstream half and a pervious zone in the downstream half. Zoning within the dams was achieved by using relatively finer-grained materials in impervious zone and coarser-grained materials in the pervious zone. Since the zoning was achieved by selective routing of construction hauling equipment within a single borrow area for the embankment fill, the distinction is slight between the grain size distribution (and probably other engineering properties) in the impervious zone and pervious zone.

After the foundation of the main dam was stripped of alluvium and highly weathered rock, a cutoff trench, typically 15 feet deep and between 10 and 20 feet wide at its base, was excavated beneath the impervious zone. Due to problems finding suitable foundation materials, the cutoff trench in the upper part of the right abutment was abandoned and an alternative trench was excavated upstream of the original one, beginning at about elevation 420 feet (NGVD, 1929). In the alternative trench less competent foundation materials were encountered in a suspected fault zone in the vicinity of the right abutment. Consequently, the cutoff was excavated deeper (as much as 45 feet) and a 38 feet deep sheet pile wall was installed below the trench between stations 1+63 and 2+01. The average depth of the piling was 14 to 15 feet and it extended from 5 feet above the trench bottom into the embankment (DWR, 1981). Below elevation 463 feet (NGVD 1929), the cutoff was excavated into relatively intact Franciscan Complex bedrock; above 463 feet, the cutoff did not penetrate entirely through the highly fractured intrusive rock.

The cutoff trench backfill and the embankment fill materials were poorly compacted to about elev. 447 feet (NGVD 1929). The DSOD inspector recognized this deficiency in a pit during construction but decided that, "considered solely from a standpoint of safety, the degree and magnitude of the condition did not merit disapproval of the unsatisfactory portion of the fill" (Wahler Assoc., 1981). The remainder of the embankment was constructed in a satisfactory manner.

After the foundation for the auxiliary dam was stripped, a cutoff trench was excavated upstream of the dam axis. The trench is 4 feet deep near the abutments and up to 9 feet deep in the channel. There is little mention about the construction of the auxiliary dam in historical records, most likely because there were few problems during construction.

Seepage on the downstream face, right groin, and upper right abutment has been observed at the Main Dam since shortly after the initial filling of the reservoir in 1938. A grout curtain was installed in 1991 to reduce seepage through the right abutment foundation. The grouting program was somewhat successful. The District currently envisions the need to construct a major downstream seepage collection system to safely collect, monitor and discharge the collected seepage.

5.0 Chesbro Dam

5.1 Cultural History

The Chesbro Reservoir was formed by the damming of Llagas Creek. The area was frequented by the Mutsun Ohlone Indians for thousands of years before the arrival of the Europeans. Today's reservoir was once part of two Mexican land grants from Mexico Governor Figueroa. The 8,927 acre Rancho Ojo de Agua de la Coche ("pig spring") was granted to Hernandez in 1835 and the 22,283 acre Rancho San Francisco de las Llagas (St. Francis of the wounds) was granted to C. Castro in 1834. The ranchos were eventually acquired by Martin Murphy, an early pioneer of Santa Clara County. By 1860, the lands had been patented to his grandson. The construction of Chesbro Dam in 1955 was part of a countywide effort by water officials to ensure adequate water supplies for the valley's growing population. The new reservoir was named Elmer J. Chesbro, a local doctor who was president of the South Santa Clara Valley Water Conservation District (the predecessor to the Gavilan Water District) at the time of its construction. The reservoir, which captures water from the Llagas Creek watershed, is used to recharge underground aquifers. This water is then pumped from wells for residential, agricultural and industrial uses.

5.2 Construction History

Chesbro Dam was designed and constructed as a zoned earthfill dam with an impervious zone in the upstream half and a pervious zone in the downstream half. Zoning within the dam was achieved by placing relatively finer-grained materials upstream in an impervious zone and coarser-grained materials in the pervious zone downstream. Since the zoning was achieved by selective routing of construction hauling equipment within a single borrow area for the embankment fill, the distinction is slight between the grain size distribution in the impervious zone and in the pervious zone. Specifically, the embankment consists of homogeneous stiff, brownish gray clayey sand with gravel. A compacted fill buttress was constructed at the downstream toe. A 4-foot thick gravel blanket drain lies between the rolled earth fill and the compacted fill buttress.

A cut off trench was constructed slightly upstream of the midpoint of the "impervious" zone. The cut off trench was excavated across the channel floor and up both abutments. Prior to grouting operations, the trench was excavated a few feet short of final grade. Grouting consisted of a single line of grout holes placed on 5- to 10-foot centers in the central axis of the trench, with a maximum grout depth of about 80 feet. After grouting, about 3 to 4 feet of native materials were removed from the cut off trench before compaction of the embankment was started.

The downstream half of the dam was constructed on unconsolidated alluvial gravel and sand deposits that were left in place. These deposits were up to about 10 feet thick. In addition, construction records describe the presence of "older alluvium" in the stream channel that was up to about 40 feet thick that consisted of boulders of serpentinite in a matrix of gravelly material. It is not clear whether this was actually an alluvial deposit or just extremely weathered serpentinite melange. The former owner, Gavilan Water District, addressed the question of liquefaction susceptibility of the foundation gravel deposits in 1980/81. A July 3, 1981 Memorandum of Design Review by D. H. Babbitt, Division of Safety of Dams, states that they concur with the comments in the consultants report (Wahler, 1981) that "...the overlying alluvium would probably be displaced downward into the contact with the underlying gravel, mobilizing shear resistance, altering the excess pore pressure in the sand zone thereby limiting movement." The referenced Design Memorandum further states that "...the dam will not be severely damaged by earthquakes." It should be noted that the dam did not experience any downstream slope movement as a result of the 1989 Loma Prieta earthquake, although there was a major relatively shallow (3-5 feet) longitudinal crack across a major portion of the crest. The materials adjacent to the crack were excavated and recompacted to the full depth of the crack. It is estimated that the peak ground acceleration at the dam site caused by the 1989 Loma Prieta earthquake was about 0.21g. The epicenter for this earthquake was about 12 miles southwest of the site.

6.0 Coyote Dam

6.1 Cultural History

Human occupation of the land surrounding the area dates back to Native American settlements of more than 3,000 years ago. The Unijaima Ohlones most likely visited the valley to hunt and fish. Pedro Fages, who had accompanied the Portola Expedition, later reported the discovery of a "Great River" on the east side of San Francisco Bay. In March of 1776, Lieutenant Colonel Juan Bautista De Anza set out from San Francisco to find this great river. His band traveled around the bay's edge to the Carquinez Straits, but returned through the Coast Range, past the Gilroy Hot Springs through what they named "the Arroyo del Coyote".

In the early 1800s, an extensive tract of land containing the present site of Coyote Lake was granted to Ygnacio Ortega by Governor Figueroa. Cattle were imported and set to graze on Ranch San Ysidro. Upon Figueroa's death in 1833, Ortega Rancho was divided among three children, with Ysabel Ortega de Cantua receiving the area which contained the present day park. This area became known as Rancho La Polka. Upon the annexation of California, the Ortega's were dispossessed of the Ranchos and the title to the lands was patented in 1860 by Martin J.C. Murphy.

In the 1890s, the Spring Valley Company of San Francisco began searching for water sources for the growing city of San Francisco. They gained title to 12,000 acres containing the Coyote Reservoir site. By 1917, the Spring Valley Company had sold most of their holdings but retained all riparian rights and 770 acres (about half of the valley's bottom). By 1913, it had become evident that Santa Clara Valley's water table was falling due to extensive well use.

In 1921, a study was completed which recommended that runoff be collected and percolated into the soil to recharge underground aquifers. It was determined that a "water district" should be formed and a system of dams should be built to regulate the runoff of the county's creeks. In

1929, the Santa Clara Valley Water Conservation District was founded and a building program was started. The Coyote Dam, one of the six original dams built within the county, was completed in 1936.

After completion of the dam, the District began to lease property around the reservoir for recreational purposes. The SCVWD title included all acreage within fifteen vertical feet above the crest of the dam. Few recreational improvements were made until the land came under control of Hall and Ada Harper in 1944. The Harpers subleased a portion of the land to a concessionaire for the operation of pleasure boats, a public dance hall, restaurant, grocery store, and arcade. These first operations were not profitable and the sublease was allowed to expire. The Harpers also subdivided one northern lakeshore and sublet the divisions as "camp sites". The following year, the new tenants began to erect cabins on their lakefront parcels. The number of cabins increased throughout the years and concessions, including a canteen, grocery store, and docks reemerged. The lease was renegotiated in 1950 and Ken and Rosemary Spencer took over. Under the Spencer's, the cabins continued to proliferate until ninety cabins lined the lake edge.

In 1956, the Santa Clara County Parks and Recreation Department was created. SCVWD then turned over management of the existing leases to the County Parks Department. The intense development along the shoreline began to cause sewage and sanitary problems in the lake. In recognition of this problem SCVWD issued a statement that the current lease to the Spencer's would not be renewed after its expiration in 1969.

In December of 1969, the County Parks Department entered into a 25-year lease for the reservoir with SCVWD. The Parks Department continued the leases on the cabins on an interim basis. Over the next decade, the Parks Department implemented improvement projects to transform the area into a regional park accessible to the public. By 1979, all of the cabins had been demolished or relocated and a majority of park facilities were in place.

6.2 Construction History

Coyote Dam was built across Coyote Creek, has a height of 140 feet above stream bed, a crest length of 980 feet, and a crest width of 100 feet at Elevation 803 feet. Construction was started in 1935 and completed in 1936. The reservoir has a storage capacity of 23,700 acre-feet at its maximum normal water surface of Elevation 777 feet, and a drainage area of 116 square miles. The dam has 23 feet of freeboard.

6.2.1 Key Issues/Geotechnical Solutions

The key issue at Coyote Dam has been the presence of the Calaveras fault in the left abutment. Few dams are knowingly constructed across an active fault. Historical records for Coyote Dam show that during the design phase of the project a difference of opinion existed between Mr. George Louderback, the geologist on the consulting board in 1934-35, and Professor C. F. Tolman, consulting geologist for the District, regarding certain characteristics of the Calaveras fault (at that time referred to as the Haywards fault) and the probable response of the dam during a major earthquake. The differences were finally resolved by agreeing that Mr. Chester

Marliave, engineering geologist for the Office of the State Engineer, would make regular visits to the site during construction in order to observe the character and attitude of the fault, whose principal active trace was thought to pass through the left (west) abutment 15° off normal to the dam axis.

The preliminary design of Coyote Dam closely followed the typical design used for other District dams of the day. Upon submitting the design to the State Engineer for approval, and recognizing that the dam would be constructed over the active trace of the Calaveras fault, the State decided that not enough attention had been paid to the possibility of fault offset within the dam foundation. As a result, the State convened a consulting board that provided the criteria and rational for the final design (Galloway, Herrman, and Louderback, 1935).

The final design of Coyote Dam is unprecedented. It represents perhaps the first dam in the world to be specifically designed to cross an active fault and to have design measures to specifically accommodate fault offset of the foundation. The design called for symmetrical upstream and downstream slopes of 4:1 below el. 718, 3.5:1 from el. 718 to el. 758, and 3:1 above el. 758. The most important aspect of the adopted design was the unprecedented width of the crest of about 100 feet and the resulting thickness of the impervious core. The core consists of a homogeneous, well-compacted, moderately plastic, sandy clay and constitutes the principal embankment material. This impervious core was designed symmetrically about the dam centerline, with a top width of 40 feet and an upstream and downstream slope of 2:1, thus resulting in a bottom width at the maximum section of 580 feet. Adjacent to the core on both the upstream and downstream sides, a free draining zone of clean gravel was placed at a slope of about 2.5:1. The gravel section is about 10 feet wide at the top near the crest and widens to in excess of 70 feet near the foundation. Heavy riprap was then placed to protect the gravels at the relatively flat slope inclinations quoted above. The extreme width of the core was specifically selected to accommodate in excess of 20 feet of horizontal offset of the Calaveras fault that had been predicted by Professor C. F. Tolman, the eminent Stanford geology professor, and Dr. G. D. Louderback, a member of the Consulting Board who also had served on the State Earthquake Investigation Commission following the catastrophic 1906 San Francisco earthquake. The overall intent of the unprecedented design was to perform as follows: in the event the earthquake created longitudinal or transverse cracking of the core, the design intended that the gravel would act as a crack stopper and the riprap would provide sufficient driving weight to force the gravel into any cracks. Although the design has a certain conceptual elegance, it has not been tested by seismic fault offset.

To reduce seepage beneath the dam, the core was seated in a cutoff trench which averaged 40 feet in depth across the main creek channel. The cutoff trench is curvilinear in plan (concave upstream) and located about 150 feet upstream of the centerline, about 80 feet downstream of the upstream core-foundation contact, at maximum section. The cutoff trench extended up the left abutment in order to fully expose the main trace of the Calaveras fault and to determine whether special treatment of the fault would be required.

Numerous construction problems beset the project; the first of these occurred in September 1935, about two months after construction work on the dam had begun. The bottom of the cutoff trench had been carried up to about el. 700 in the left abutment when about 20,000 cubic yards of material slid into the cutoff trench. The top of the slide varied from elevation 767 to 794 with the approximate center of the top of the slide about 190 feet upstream from the axis of the dam. Although the slide was not well documented, several inspection reports indicated that cleanup of the slide debris was accomplished without further problems.

The performance of the embankment has been closely monitored since the facility was put into operation. One incident worthy of note was the severe spillway damage that occurred after major spillway discharges following heavy rains in March 1937. It took approximately three years to repair the spillway completely; most of the major damage occurred to a concrete-lined stilling basin downstream of the present spillway terminus. Deterioration and failure of the reinforced concrete spillway floor and wall slabs was also a major problem for the first several decades following construction. This earlier problem was related to swelling of the Cretaceous rocks (Berryessa Formation) within which the spillway was formed. Cuts as deep as 130 feet were required to form the spillway (Tepel, 1985).

Siltation of the reservoir required that the intake structure be raised on several occasions. In the late 1980s, the depth of reservoir silt required that the inlet again be raised. The seismic design requirements were so rigorous that the District decided to abandon the original 50-inch diameter outlet pipe (it was tremmie filled with a specially designed bentonite-cement mixture and then grouted) with a new outlet tunnel located entirely in bedrock between the dam and spillway east of the presumed active trace of the Calaveras fault. Nevertheless, the tunnel required special design to accommodate a potential 18-inches of offset on the Right Abutment Fault. The tunnel was built oversize (10-feet) in order to facilitate visual inspection. As a part of the new tunnel studies, the consultant was required to thoroughly evaluate the historical movements of the dam. Long-term vertical and horizontal deformation of the dam has been detected through the analysis of survey monuments located on the dam crest. Based on these studies, several hypotheses were developed to explain the source(s) of the deforming stresses including:

- active fault creep on one or more strands of the Calaveras fault:
- settlement of the dam due to normal consolidation of the embankment materials;
- settlement induced by strong shaking during local earthquakes on the Calaveras fault; and
- downslope movement of deep-seated landslides in the left abutment area (ESA, 1989).

Following construction of the new outlet, a major study of the left abutment landslide, probable location of the Calaveras fault, and evaluation of historical dam movements was undertaken by the District (Geomatrix, 1994). These studies concluded the following:

(1) right-lateral displacement attributable to creep on the Calaveras fault has occurred over a crest distance of about 300 feet at a relatively constant rate of 0.36 inch/year (9 mm/yr.);
(2) in addition to ongoing settlement, the embankment experienced a maximum settlement of 4 inches (10 cm) during the April 1984 Morgan Hill earthquake;
(3) since about 1970, monuments on the dam crest indicate that an active strand of the Calaveras fault with an average rate of right-lateral creep of about 0.47 in/yr. (12 mm/yr.) passes beneath the left abutment west of the silica-carbonate knob that is partially encapsulated by the left side of the dam; and
(4) based on the results of two inclinometer casings, the existence of a major landslide within the left abutment was inconclusive although possible future movement will be evaluated by monitoring the inclinometer borings.

7.0 Guadalupe Dam

7.1 Cultural History

The Guadalupe Reservoir area shares essentially the same cultural history as Almaden Dam (see Section 2.1 of this appendix).

7.2 Construction History

Guadalupe Dam was designed and constructed as a zoned earthfill dam with an impervious zone in the upstream half and a pervious zone in the downstream half. Zoning within the dam was achieved by using relatively finer-grained materials in the impervious zone and coarser-grained materials in the pervious zone. Since the zoning was achieved by selective routing of construction hauling equipment within a single borrow area for the embankment fill there does not appear to be a significant distinction between the soil properties of the impervious zone and pervious zone. Based on laboratory testing results (DWR, 1982; Wahler and Associates, 1981) the upstream zone and downstream zone have similar average dry unit weight (124.9 pcf versus 122.3 pcf, respectively), average natural water content (13.3% versus 14.5%, respectively), and permeability (4.6×10^{-9} cm/sec versus 5.2×10^{-9} cm/sec, respectively). For analytical purposes, the embankment should be considered to consist of a homogeneous stiff, brownish gray clayey sand (SC) with gravel. A cut-off trench was constructed slightly upstream of the midpoint of the "impervious" zone.

Foundation earth materials beneath the dam are sheared and closely fractured shales and sandstones of the Jurassic-Cretaceous Franciscan Complex. Clay soil up to 20 feet thick was encountered at the base of the left abutment and clay and gravel of an alluvial terrace was encountered in the channel area and beneath the lower right abutment. The soil materials described above encountered within the dam foundation were removed and used as suitable embankment fills, as appropriate.

8.0 Lenihan Dam/Lexington Reservoir

8.1 Cultural History

The Los Gatos Creek basin, which originally was referred to as the Lexington basin and extends from the valley floor back to the ridgeline separating Santa Clara Valley from Santa Cruz county, is believed to have been used by the Ohlone Indians for thousands of years as part of a trail corridor to the coast and Santa Cruz area. This trail was used for hunting access to seasonal camps and for trade with neighboring tribes. The trail was later used by the Franciscan missionaries connecting Santa Clara and Santa Cruz missions.

One of the first American settlers in the Lexington basin was a sea captain named Willard Hanks. In 1838, he married a Mexican girl named Isabella in order to be able to stay in Alta California. He later moved from San Jose to the foothills of the Santa Cruz Mountains to build a cabin in what is now the Lenihan (Lexington) dam area. In 1847, Hanks sold an equal share in water and lumber rights of his property to Isaac Branham for the purpose of erecting a saw mill. In 1848, Zachariah Jones approached Hanks and Branham with a proposition to build a dam across Los Gatos Creek, two miles above Los Gatos, to further develop a logging camp. He later bought out Branham and Hanks.

The earliest industry in the mountains was logging. Lumber camps sprang up and entire slopes were denuded. From the Lexington area, loggers supplied the timbers for the miles of timber shoring required in the tunnels in the Almaden Quicksilver Mines. Zachariah Jones' business and lands grew. Jones petitioned the County Board of Supervisors to improve the road but the project was dropped. In 1857, the Santa Cruz Gap Turnpike Joint Stock Company was formed and adopted a survey to construct a 12 to 20 foot wide road from Los Gatos up the west side of Los Gatos Creek Canyon to the County line. While Jones filed suit, work began along the old Franciscan Trail and the toll road opened in 1858.

Jones won his suit against the Turnpike Company, but he eventually sold his property to John Henning, who founded the town of Lexington. Henning named the town after his hometown in Missouri. Lexington became the largest settlement in the Redwood District which included Alma, Saratoga, and Los Gatos. The Toll Company Charter expired in 1877, but tolls stayed in effect. The residents of the Redwood District petitioned the Board of Supervisors to declare the road a public highway and the Board agreed.

In November 1861, oil was discovered near Moody Gulch. An oil field was developed by Charles Ellis and C.C. Iver, leased from D.B. Moody, yielding 160 barrels a month. Rumors of cinnabar and gold brought more people to the area, and a stage route opened spawning many new towns. The competitive and dangerous stage lines gave way to railroad construction. By 1877, Chinese laborers were clearing a railbed up Los Gatos Creek Canyon and in 1878 the Southern Pacific Railroad reached Los Gatos. Narrow gauge rail lines were laid to Lexington and Alma. By 1880, a hotel had been built in Alma and the town included a postal and telegraph office.

In 1887, Leland Stanford funded the transition of the narrow gauge to a standard gauge rail line. By 1895, the standard gauge third rail had been laid from Santa Clara to Los Gatos. Efforts were made to transition the entire line over the mountains. This involved the boring of new tunnels, the first of which was completed near Alma in 1903. In 1919, Highway 5 (now Highway 17), connecting Los Gatos and Santa Cruz was completed with the road being paved in 1921. By the 1930s, even though the trains were packed with passengers and freight, the line was not profitable. Competition with the automobile and the advent of WWII forced the line to close in 1940. In 1939, the road was replaced with a four lane highway and renamed Highway 17, bypassing many of the historic mountain resorts and communities. It is interesting to note that the train tunnels were dynamited closed in 1940 when it was thought possible that an invading army could access the tunnel on the Santa Cruz side and have a straight shot access to the Santa Clara Valley. The tunnel was nearly exhumed following the 1989 Loma Prieta earthquake when seismologist from the U.S. Geological Survey wanted to enter the tunnel to determine the nature a magnitude of offset on the San Andreas fault where it passes through the tunnels. The work, however, was never funded and the tunnel remains closed.

In its heyday, Alma was an important stage coach stop, lumber mill site and train stop. The town was located near today's Alma fire station, three miles above Los Gatos. The town was an important shipping point for the many fruit growing areas of the mountains and also functioned as a resort. Prunes and grapes, along with beekeeping, were the main agricultural efforts of the area.

The town of Lexington was situated closer to Los Gatos, near where Black Creek enters the reservoir today. When the lumber industry was booming in the Santa Cruz Mountains, Lexington was a thriving town. The turnpike went through town, which included a hotel, livery stable, and blacksmith shop. J. S. Beardsley owned over 300 acres of what is Lake Canyon today. Once the railroad was built, however, most of the business moved to Alma.

The construction of Lexington Dam and reservoir in 1952 was part of a countywide effort by the Santa Clara Valley Water District to address diminishing water supplies in the early part of the 20th century. The reservoir was appropriately named for the historic town of Lexington, which along with the village of Alma, was located where the reservoir now exists. In 1958, the Santa Clara County Parks and Recreation Department leased the reservoir for recreational purposes. The County added several parcels in 1989 and 1990 to create today's 960 acre park. The name of the dam was changed from Lexington to Lenihan in 1996 after James J. Lenihan, long-time retired District Board member. The Lexington Reservoir name was not changed.

8.2 Construction History

Lenihan (formerly Lexington) Dam is a rolled earthfill dam that was constructed in 1952-53. It is located on the east side of State Highway 17 in Santa Clara County, about 1½ miles south of the Town of Los Gatos. The dam, which impounds water in Lexington Reservoir for the purpose of groundwater recharge, was constructed across Los Gatos Creek. The water released from the dam is generally conveyed through the District's Vasona Dam and Reservoir, located about 2 miles north on Los Gatos Creek, to recharge ponds located on the west side of Santa Clara Valley.

The structural height of the dam is about 194 feet as measured from the lowest point in the foundation (Elevation 479 ft) beneath the axis to the crest of the dam. The maximum height as measured from the lowest point at the downstream toe (Elevation 466) to the dam crest is about 207 feet. The dam crest, at an average Elevation 673±, is about 40 feet wide and 810 feet long. The crest is not flat, but cambered in both directions from the approximate middle to the left and right abutments. Also, due to the presence of the Alma Bridge Road roadway across the crest, the crest slopes across the axis from the middle of the roadway towards both the upstream and downstream faces for drainage purposes. The upstream face is inclined at 5.25 horizontal (H):1 vertical (V) (i.e. 5.25H:1H) from the crest down to Elevation 648, and 5.5H:1V from Elevation 648 to the upstream toe; however, within the original Los Gatos Creek channel, the upstream shell terminates at Elevation 553 and merges with a 25-to 60-foot-wide rockfill toe that is sloped at 2.5H: 1V down to the upstream toe at about Elevation 493. The downstream slope is inclined at 2.5H:1V from the crest down to Elevation 648, 3H:1V from Elevation 648 to Elevation 553, and 2.5H:1V from Elevation 553 to the downstream toe which, at the maximum section, is at Elevation 463.

An ungated Ogee crest spillway is located in the left (west) abutment. The spillway crest is at Elevation 653, with a crest length of about 150 feet, and a spill-length of about 925 feet. A hydrologic study was performed by the District (SCVWD, 1993). The principal conclusions presented in the District hydrologic study are as follows: 1) the mean annual precipitation at the site is about 40 inches; 2) the 72-hour Probable Maximum Precipitation is estimated to be about 61.8 inches; and 3) the Probable Maximum Flood is 43,500 cubic feet per second (cfs) at a maximum pool of Elevation 670.7. The reservoir capacity, with water at the spillway crest, is about 19,834 acre-feet.

The Lenihan Dam low level outlet pipeline is approximately 1,573 feet long. It extends from a gated intake structure upstream of the dam (centerline of the original gate was at Elevation 566.0, but it was subsequently raised due to reservoir siltation) to a free discharge structure (centerline of outlet at Elevation 469.3), near the downstream toe of the dam. The original outlet pipe is composed of 1,333 feet of 50-inch diameter steel pipe encased in a nominal 12-in. thick reinforced concrete, and 240 feet of 48-inch diameter reinforced concrete pipe at the D/S end, which is also encased in concrete. The first 242 feet of the steel pipe (the new extension that was installed in 1988) is 3/8-inch thick, and the remaining 1091 feet (downstream) is 5/16-inch thick, except for the buckled sections that have been replaced with 3/8-inch thick steel plate. The 5/16-inch thick pipe is part of the original outlet pipe, constructed in 1952. The outlet pipe has experienced several partial collapses of the steel liner due to excess external pressure combined with vacuum pressures (RLVA, 1999), corrosion, and out or roundness, and has recently been repaired.

The dam has an interesting construction history. The fill material to construct the core and upstream dam shell came from borrow sources on the upstream side of the dam. The lower portion of the core, up to about elev. 590, was derived from very clayey material that had been washed down the creek located on the east side of the reservoir near the parking lot. The upstream shell and the upper core were derived from excavation within the exposed Franciscan material just upstream of the upstream toe. As the dam was being constructed, the Contractor had difficulty in generating sufficiently clean rockfill for the horizontal drain that underlies the downstream shell up to elev. 483, and the inclined drain materials. As the delays in producing the required drain material continued, even while the core and upstream shell were being constructed, the State engineers became concerned. Since the spillway was behind construction, they were concerned that in case of a heavy rainstorm, the reservoir could fill and overtop the dam. The State allowed the Contractor to finish a major section of the dam prior to the oncoming wet season without constructing any of the inclined drain. The following dry season, when the horizontal and inclined drain materials were available, work continued with constructing the downstream portion of the dam. The downstream shell was primarily derived from required excavation of the spillway. The downstream shell material is only slightly coarser that the upstream shell.

As a result of the unique delay in constructing the inclined drain, it was subsequently discovered during the 1998 emergency investigation that the drain was not constructed exactly as shown in the As-Built drawings. This was revealed by hand written notes from 1952 Dresselhaus that were found in the DSOD files. The reason for this was deduced as follows. Since the entire upstream shell and core were constructed ahead of the downstream shell, the contractor constructed the downstream edge of the core at an inclination of about 1.25:1 (H:V), which was much flatter than the intended inclination of 0.9:1 (H:V). Also, the following dry season, prior to beginning the construction of the inclined drain, the contractor apparently constructed a wedge of transition material between the lower core and the inclined drain. The transition zone was discovered by careful logging of the drill cuttings during placement of piezometer LVP-10, which is located near the downstream corner of the core. Based on the thickness of the inferred transition material from this boring, and the knowledge that the core had been originally constructed with a 1.3:1 (H:V) slope, it was deduced that the transition zone has an inclination on the upstream side of about 1.5:1 (H:V), an inclination on the downstream side of about 0.9:1 (H:V), and a top width of about 65 feet at elev. 590. Apparently, as the inclined drain was being constructed, the Contractor made a sizable cut into the existing core to get good bonding prior to placing the sandier transition material. Once the transition was completed, the narrow (~2 ft. wide) inclined drain was then constructed at the originally intended inclination of 0.9:1 (H:V) between the transition and downstream shell. Construction photographs show good construction techniques of using a spreader box and keeping the inclined drain ahead of the shell to prevent contamination. Although the exact configuration of the transition material is not known, it is interesting that the recently installed instrumentation clearly shows the phreatic surface being intercepted by the transition zone before it reaches the inclined drain.

It is also interesting to note that the San Jose Water Company (SJWC) has water lines that cross the east side of the reservoir, up the upstream right (east) groin, and then down the east downstream groin of the dam. These pipes carry water that has been released into Los Gatos Creek from the SJWC's Austrian Dam and Reservoir, a 6,200 ac.-ft. facility located just north of Summit Road in the headwaters for Lexington Reservoir. Near the crest of Lenihan Dam, the SJWC pipes turn and head west across the crest where they supply the SJSC's Montevina Water Treatment Plant. The tank for this facility can be seen in the aerial photo of Lenihan Dam presented in Appendix A.

Also, as mentioned previously, the Alma Bridge Road crosses over the crest of Lenihan Dam. With the spillway located on the left (west) side of the dam, it was necessary to construct a bridge across, and just upstream, of the entrance to the spillway weir. The bridge received minor damage during the 1987 Loma Prieta earthquake and has subsequently been repaired. Although the piers supporting the bridge are located in the spillway entrance area, they do not have a significant impact on the spillway hydraulics or flow characteristics during spillway discharges.

9.0 Stevens Creek Dam

9.1 Cultural History

The Ohlone Indians lived in and around Santa Clara Valley for over 3,000 years prior to the arrival of the Europeans. The Guemelento Ohlone Indians inhabited the upper reaches of Stevens Creek where a large village, known as Ritocsi, was located. The Ohlones of this area were named the San Jose Cupertino Indians by the Spanish Missionaries. These people most likely used the present day Stevens Creek County Park areas for hunting and gathering food.

On March 25, 1776, the Feast day of San Josepha de Cupertino, an expedition led by Colonel Juan Bautista de Anza camped on Stevens Creek which they named Arroyo San Josepha de Cupertino. Padre Font, who accompanied the party, described the area in his diary as... *"Having good water and much firewood, but nothing for settlement because it is among the hills very near the range of cedars I mentioned yesterday and lacks level lands."*

The creek and reservoir were named for Captain Elisha Stephens who lived on the creek around present day Blackberry Farm between 1848 and 1864. Stephens was born in South Carolina in 1804 and learned the trade of blacksmithing in Georgia. He later traveled over much of middle and western American. In the spring of 1844, he was selected as Captain of the Stephens-Murphy-Townsend party headed for California. This "wagon train" of 50 people and 11 wagons would be the first party to cross the Sierra Nevada. With the guidance of a Paiute Chief they called Truckee, the party followed a river to a lake which would later become known as Donner Lake. The wagons reached the summit on November 25, 1844. The party left behind a crude log cabin at the lake which would be used two years later by the Donner-Reed Party.

In 1848, Stephens purchased a 160-acre homestead he called Blackberry Farm on the Arroyo de San Joseph Cupertino (which later became known as Steven's Creek). He farmed the property growing Mission grapes, fruit trees and blackberries. He was well known in the area for killing and eating rattlesnakes. In 1859, he purchased an additional 155 acres. By 1864, Stephens had sold his property, claiming that "it's gittin' jist too crowded, too durn civilized 'round here". He relocated to Kern County where he started a small ranch in what is now Bakersfield. Stephens died at the age of 83 and is buried in Bakersfield.

Catherine Duncan bought part of what is now Stevens Creek Park in 1869 from the U.S. Government (under the Homestead Act) and later sold 160 acres to Santa Clara College. The 320 acres which originally comprised "Villa Maria" were purchased as a farming investment and vacation spot for the College. A Chapel, Villa House, winery and various barns were built. The Jesuit Brothers were responsible for planting the eucalyptus and pines which can still be seen lining the roadways today. The Villa served many purposes which included a retreat for Catholic men and a quarantine camp for students and faculty of the college. The college sold the property in 1945. The new owner ended up selling most of the property for quarrying operations. The Villa Maria chapel was torn down around this time in 1947.

The County purchased 400 acres in the upper reaches of the canyon (near Mt. Eden Road) in 1924. Under the direction of the Public Works Department, this became the first County park. In 1935, the Santa Clara Valley Water Conservation District purchased 171.5 acres of land northwest of the county's 400 acres to develop a reservoir. The Stevens Creek dam, one of the original six dams built in the county, was completed in 1936. The County's 400 acres were eventually transferred to the newly created Parks and Recreation Department in 1956. Additional parklands have been added since the 1960s.

9.2 Construction History

Foundation earth materials beneath the dam consist of alluvial terrace deposits (alluvium) underlain by rocks of the Santa Clara Formation. The alluvium is present over only a portion of the foundation. Although not shown on the "As-Built", it is likely alluvium was left in-place in the main channel section of the former creek channel, but was not left in-place in the abutments. According to the boring logs developed in 1982, Wahler field geologists described the alluvium as ranging from loose to medium dense sand, with less than 15 percent fines, to soft/firm clayey sand with gravel. The alluvium ranges from about 10 to 15 feet thick and is present beneath the central and right abutment areas of Stevens Creek Dam. The Santa Clara Formation consists of pebbly sandstone and claystone. A cutoff trench was constructed at about midpoint of the impervious zone. The dam embankment consists of an "impervious" zone in the upstream portion and a "pervious" zone in the downstream portion of the dam. A concrete slab provides upstream slope protection.

In 1978 and 1982, Wahler performed seismic safety evaluations of Stevens Creek Dam using dynamic finite element analyses methods and determined that a M 8.3 design earthquake on the nearby San Andreas fault could cause excessive shear displacement of the upstream shell and probably result in significant lose of freeboard. Based on Wahler's remedial design recommendations (1984 report), new stabilizing sections were added to the dam in 1985. Stabilizing section construction consisted of placing a buttress fill over the upstream toe of the dam to about el. 490 feet. The purpose of the buttress fill is to provide additional earthquake resistance to both the upstream and downstream slopes. Also, raising the dam crest about 10 feet with new buttress fill on the downstream face provided additional freeboard to combat earthquake-induced shear displacements. The buttress fill is about 100 feet wide at elev. 490, and between 20-40 feet thick, over most of the upstream shell. The buttress fill is about 20 feet thick over most of the downstream shell. An inclined filter/drain was placed between the original downstream face of the dam and the new buttress fill. The filter/drain was designed to prevent the downstream buttress from becoming saturated, and to provide a "crack-stopping zone" near the crest of the dam. With the addition of the upstream and downstream buttresses, the zones of the dam now include the original "impervious" and "pervious" zones, and new downstream fill Zone 1A, upstream buttress Zone 1B, and filter/drain Zone 2.

10.0 Uvas Dam

10.1 Cultural History

The Uvas Reservoir area was frequented by the Mutsun Ohlone Indians for thousands of years before the arrival of the Europeans. Today's reservoir was once part of the 11,093 acre Rancho Las Uvas (meaning "Ranch of the grapes"), granted by Mexican Governor Alvarado to Lorenzo Pineda in 1842. The land later became patented under the American courts to Martin Murphy, an early pioneer of Morgan Hill and Gilroy. The lush valley was used for agriculture including plums. At the turn of the century a woman named Minnie Kell, or "Ma Kell" ran a campground in the Uvas Creek Valley, featuring a swimming hole, baseball diamond and dance floor. The construction of Uvas Dam by the Gavilan Water District in 1957 was part of a countywide effort by water officials to ensure adequate water supplies for the valley's growing population. Water from the reservoir is used to recharge underground aquifers, which will later be pumped from wells for residential, agricultural and industrial uses. In 1960, the County Parks and Recreation Department began recreational operation of the reservoir.

10.2 Construction History

Since construction in 1956 and 1957, Uvas Dam has had recurring seepage problems. Several attempts have been made to mitigate and control seepage flow through the embankment, foundation, and abutments. These efforts include installation of several drains, grout curtains, and 6-inch diameter relief wells (RW-1 through RW-4). The relief wells are now used for monitoring piezometric levels.

The dam was constructed on Jurassic-Cretaceous Franciscan Complex rocks. The rock types near the dam include basalt, greenstone, limestone, chert, shale, and sandstone. The bedrock underneath the dam is blanketed with Quaternary deposits, which have been interpreted as consisting of artificial fill, colluvium, channel, terrace, and landslide deposits. During construction of the dam, a grout curtain was installed along the centerline of the cutoff trench. In May 1958, a second grout curtain, consisting of 18 grout holes, was placed between the spillway weir and the left end of the dam. In 1996, the District installed a double-row grout curtain in the foundation bedrock of the right abutment.

Uvas Dam was constructed as a zoned earth dam using Franciscan-derived material. As-built drawings show upstream and downstream "less pervious" zones and a central core that extends partly upstream identified as "select impervious". An inclined chimney drain at the downstream face of the core and a horizontal blanket drain along the downstream toe of the dam were also incorporated into the design. Previous investigations found that the embankment materials are generally gravelly, and range from weathered bedrock fragments to gravels to clayey sands and gravels. After construction, several drains were installed within the embankment to reduce pore pressures. These included horizontal drains near the left abutment, a downstream toe drain, several 6-inch diameter monitoring wells (also referred to as "relief wells") along the downstream berm, and a chimney drain at the downstream face of the core and interceptor drain along the downstream toe.

11.0 Vasona Dam

11.1 Cultural History

The origin of the "Vasona' has its roots in local folklore. Frazier O. Reed II, great-grandson of the co-leader of the Donner Party, said his maternal grandfather, Albert August Vollmer, was the man who put the name on the area before the turn of century, long before there was a Vasona Lake County Park.

Albert Vollmer moved his family from Michigan in 1887, and brought them to the Santa Clara Valley. The Vollmers settled on a 46-acre prune ranch on Pollard Road about three miles from Los Gatos.

The oldest of the Vollmer children, Agnes, worked in San Jose for the Sunset Telephone and Telegraph Company (which became the Pacific Telephone Company in 1907) and she commuted by train from Los Gatos. Every morning her father took Agnes to Los Gatos in his buggy and every evening he picked her up. Since the train passed about a mile from the ranch, he asked the Southern Pacific Railroad if a flag stop could be established.

The Southern Pacific agreed, telling Vollmer he could name the stop because he had requested it. That stop became the "Vasona", named after a pony Albert August Vollmer had as a child.

11.2 Construction History

The initial geologic report regarding the Vasona Dam site was written by the late Professor C. F. Tolman of Stanford University in August, 1934. The original investigation consisted of five boreholes which were located at equal intervals across Los Gatos Creek. It was concluded from the exploratory work and the logs of several nearby wells that an extensive horizon of impervious "blue clay" existed at depths varying from 10 feet under the existing streambed to 24 feet in the right abutment and 20 feet in the left abutment.

The dam was designed by Fred H. Tibbetts, Chief Engineer for the Santa Clara Valley Water Conservation District, predecessor of the present District. The design consisted of a concrete slab and overpour section flanked by earth embankments with an upstream slope inclined at 2(H):1(V) and a downstream slope at 3:1. In order to effectively prevent leakage under the dam, a cutoff trench was included in the design. It was located beneath the upstream toe and has an average depth of 20 to 22 feet and a bottom width of approximately 10 feet at the contact with the "blue clay." The cutoff trench was backfilled by "puddling" materials having a high clay content that were derived from a borrow source within the reservoir. This construction technique was common in the 1920s and 1930s and consisted of constructing a surrounding dike and flooding the area between the dikes with water and clay. This process was repeated until the desired thickness of clay was achieved.

The basic earth dam section consists of an upstream zone of fine material selected from the borrow area, with downstream zones of coarse materials selected from borrow areas (described in construction inspection reports as "gravel fill") and unclassified materials from the cutoff trench excavation. Materials from the foundation stripping, which apparently occurred only in the upstream foundation area, were placed at the downstream toe.

Compaction of fine materials in the upstream zone was accomplished by means of a tamping roller, with specifications requiring 9-inch compacted lifts. Materials in remaining portions of the dam were placed and spread in 9-inch lifts, with compaction achieved only by action of the hauling units and tractors which spread the materials.

The spillway section is approximately 220 feet wide and is apparently founded on a medium dense to very dense sandy gravel horizon which overlies the "blue clay" of the Santa Clara Formation. The original spillway consisted of eleven radial gates, nine of which measured 5 by 21 feet and two of which measured 13 by 10 feet. The maximum controlled water surface with the gates closed was elev. 300 feet. In 1975-76, however, as a result of concern over their performance and continued use in the event of a major earthquake, all of the radial gates were removed from the spillway. Currently, therefore, the maximum stored water surface possible at Vasona Dam is elev. 295.

Vasona Dam, one of the Districts original six dams built in 1935, has been significantly altered since its construction. Prior to the construction of Austrian Dam in 1950 (owned and operated by the San Jose Water Company) and Lexington (now Lenihan) Dam in the 1953, Vasona Dam provided the major flood control on Los Gatos Creek. Since the construction of both Austrian Dam and Lenihan Dam, a significant amount of the rainfall falling within the Los Gatos Creek watershed is captured and stored behind these two major dams. As a result, the significant flood capacity in the lower Los Gatos Creek area that was originally provided by Vasona Dam is no longer needed. As a result, the majority of the smaller (upper) radial gates at the dam were permanently removed in 1971. The removal of these gates (which were located on the west side of the larger lower radial gates) caused a reduction in lake capacity from 750 acre-ft to 400 acre-ft, and permanent lowering of the maximum reservoir elevation of Vasona Lake from 302.6 feet to 297.6 feet (NAVD 1988). The larger lower radial gates are still functional and used for flood releases if needed. The primary use of Vasona Dam today is to provide control of water released from Lenihan Dam to the downstream seepage recharge ponds operated by the District and for recreation at Vasona Reservoir.

APPENDIX C

GEOLOGIC AND SEISMIC ISSUES AT DISTRICT DAMS

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1.0 Geologic Issues

This appendix describes how geology directly affects District dams, and specifically addresses certain engineering geologic issues that currently impact the dams. It is not the intent of this appendix to provide detailed discussions of geologic conditions at each dam. Rather, the focus is on pertinent geologic issues including: 1) specific geologic units encountered in the foundation bedrock that possess unique issues of concern to dam safety; 2) response of the dams to specific local/regional tectonics; 3) geologic conditions that affect potential foundation seepage at specific dams; 4) fundamentals of seismicity and its affects on District Dams; and 5) how significant earthquakes in the San Francisco Bay could impact dam safety in the future. Not all of the dams are significantly affected by each of these geologic issues, and thus are not included in each discussion.

2.0 Importance of Geology

A discussion of the safety of District dams is incomplete without considering the surrounding geologic environment that comprises the San Francisco Bay Area. The dams are literally a part of the geologic terrain because they were constructed of earth material derived from local sources and are founded in, and supported by, local bedrock. Consequently, the construction history, year-to-year performance, and long-term behavior of the dams are intimately connected to the underlying rocks. These rocks make up a rich geologic tapestry that developed during the last 160 millions of years as a result of tectonic processes that continue to the present.

The geology of California Coast ranges in general, and the San Francisco Bay Area in particular, are world famous, primarily because 1) the oldest rocks, known as the Franciscan Complex, formed as a subduction-prism as a result of Plate Tectonic processes, 2) the San Andreas fault was the one of first recognized major active tectonic features in modern history affecting a large area and experiencing significant strike-slip movement, and 3) the Bay Area is the most active tectonic region in the contiguous US and has experienced many of the its largest and most historic earthquakes.

3.0 Geologic Setting

The San Francisco Bay Area is situated in the Coast Range geomorphic providence, which is a wide soft plate boundary between the Pacific plate to the west and the comparatively rigid Sierra Nevada-Great Valley subplate of the North American plate to the east. This soft portion coincides with the lateral extent of a prism of weak subduction-disrupted rocks of the

Franciscan Complex. Consequently, the Coast Ranges developed in response to intense plate deformation concentrated in the weak Franciscan terrane, leaving more rigid adjacent terranes relatively intact (Thompson, 2000). This explains why "bedrock" in the Bay Area is notorious for having soil-like properties (i.e. very weak and weathered), unlike the "ringing" rock of the Sierra granite. It is the relatively softer and soil-like properties of the local bedrock that provide a challenge to geotechnical engineering.

Topographically, the Coast Ranges have a mean height of about 1500 feet (0.5 km) and a width of about 75 miles (120 km). Although the Coast Ranges were being intensely folded and thrust-faulted at various intervals of the geologic past, their present topographic expression is surprisingly young. This is evident by the presence of steep mountain fronts involving young Pleistocene sedimentary formations that are bounded by folds and predominantly reverse faulting (with some strike slip faulting).

Most of the plate motion between the Pacific-Sierra Nevada plates is accommodated by strike slip movement along the San Andreas fault-system; however, roughly 5 to 10 % of the motion can be attributed to convergence in the form of folding and reverse or thrust faulting (Thompson, 2000). This relatively small component of generally northeast-southwest oriented compression accounts for the structural and topographic relief (mountain building) that is still occurring within the Coast Ranges. If the motion along the San Andreas fault-system was perfect strike-slip with respect to accommodating plate motion, then the Coast Ranges would not have formed. The fault would have continuously realigned to exactly fit plate motions so that the plates would glide past each other without the need for folding and reverse faulting. The presence of more than one active fault trace of the San Andreas system, which includes the Calaveras and Hayward faults to the east and the San Andreas to the west as it passes through the San Francisco Bay Area, appears to coincide with a major bend in the fault near Hollister when viewed from south to north.

Until recently, local geologists didn't fully appreciate the magnitude of convergence in the San Francisco Bay Area. A possible repercussion to District dams is that even if critical structures are not located directly on a specific active fault trace (although Coyote Dam is the exception) structures still have the potential to experience deformation in response to local and regional tectonic forces, albeit at a relatively minor rate of several millimeters per years.

4.0 Variations in Local Bedrock

4.1 Franciscan Complex

All District dams, except Stevens Creek and Vasona Dams, are founded on Franciscan Complex bedrock. As mentioned above, these represent the oldest rocks found in the Bay Area, ranging in age between 80 millions to 160 millions of years, corresponding to the Jurassic-Cretaceous period; better known as the age of the dinosaurs. During this time period of California's geologic history dinosaurs were not present because California did not exist as dry land; it was occupied by a subduction zone complex involving the incorporation of sea floor rocks from the west onto the continental margin to the east along an active subduction plate margin. The "western" coast of the Pacific Ocean during this time was about the western edge of the current Rocky Mountains.

The Franciscan Complex is world famous as 1) a sequence of metamorphic rocks that uniquely formed under high pressure and low temperature conditions characteristic of a subduction zone and 2) containing mélanges, which are chaotic geologic units that have a sheared matrix and contain a variety of blocks. The sheared nature of the mélange is directly related to its formation during subduction. The Franciscan Complex occurs in belt-like units with northwest-trending long axis. Each belt consists of stacks of thrust or nappe sheets, which are relatively coherent rock units 1-2 km thick (Wakabayashi and Hengesh, 1995). The relative order of stacking is related to the progressive incorporation of individual nappes, generally becoming younger downward, representing progressive offscraping or underplating of units in the subduction zone.

The Franciscan Complex developed in a subduction zone during the late Mesozoic underflow of the Pacific oceanic crust beneath the continental plate. The mélange units are regarded as a product of tectonic churning and mixing of trench deposits and material scraped off the descending oceanic slab. According to Plate Tectonic Theory, the North American plate (to the east), Pacific plate (to the west), and Juan de Fuca plate (to the north) all meet at the Mendocino triple junction, located about 25 miles off the coast and due west of Eureka, California. This junction also represents the southern extension of the very large Cascadia subduction zone that lies immediately off the coast of Oregon and Washington. The Pacific plate has been moving north-northwestward relative to the North American plate at a rate that could have been as much as about 6 cm/yr for the past 2 million years and probably less than 1.2 cm/yr for the past 23.5 million years.

4.2 Serpentinite

Serpentinite is believed to be have been intruded into rocks of the Franciscan Complex shortly after initial formation or implacement of the subduction prism. Thus, serpentinite is considered to be a separate and younger bedrock unit than the Franciscan Complex. Serpentinite forms originally as "ultramafic" rock that has subsequently been "serpentinized". Ultramafic refers to rocks that contain dark, heavy minerals and were formed originally as oceanic crust at the Mid-Pacific Rise, a Plate Tectonic spreading zone near the center of the Pacific Ocean. The earth's crust is formed by Plate Tectonics processes at spreading zones, located typically beneath the oceans. At the spreading zones, material is coming up from the earth's magma and spreading out along the earth's sea floor. The earth's crust is consumed in subduction zones, usually located at continental margins.

Serpentinization is the hydrothermal alteration of the ultramafic rock after its formation (due to the action of heat and mineralized ground water) that changes the original ultramafic minerals into serpentine minerals. In the geologic past, along the Pacific Coast, the action of plate tectonics has resulted in subduction of the oceanic crust both beneath and onto the North American Continent, specifically the California coast. The result is that serpentinite is associated with tectonic mixtures (mélanges) and shear zones. Exposures of serpentinite indicate that some type of faulting or shearing has occurred. If one looks closely at serpentinite one will see that rock surface is shiny and polished, which are features that directly relate to the tectonic origin of the rock. In mélanges (tectonic mixtures) the serpentinite blocks can vary from cobbles to the size of small homes, or larger. While serpentinite can be mixed with various types of rocks in a mélange, one can also find areas that are serpentinite mélanges, which are composed solely of serpentinite. Serpentinites are usually associated with mélanges and shear zones of the Franciscan Complex and are notorious for their very weak strength.

4.3 Silica-Carbonate

Hydrothermal alteration is a geologic process that helped create the New Almaden mining district, one of the richest quicksilver (mercury) mining areas of the New World. This process generally involved the introduction of chemically-rich hot solutions that changed serpentinite into a chemically- and texturally-different rock known as "silica-carbonate" because it contains silica (quartz) and carbonate (limestone). The silica-carbonate is the host rock for mercury in the form of cinnabar in the New Almaden mines.

Hydrothermal alteration and preferential dissolution of the carbonate resulted in the formation of cavities (much like the formation of caves in limestone), commonly found in silica-carbonate rocks. The presence of cavities makes silica-carbonate a poor quality rock for dam foundations because the cavities allow flow of water in the form of seepage. As discussed below, construction of Almaden Dam was significantly affected by the presence of poor quality silica-carbonate bedrock in the left abutment. Almaden Dam is the only District dam thus affected.

5.0 Local and Regional Tectonics

5.1 Chesbro and Uvas Dams

McLaughlin and others (2000b) mapped what they term the "Uvas Antiform" as a structural feature developed in Franciscan Complex bedrock in the region between Chesbro Reservoir to the north and Uvas Reservoir to the south. Chesbro Dam lies along the northern limb and Uvas Dam lies along the southern limb of the antiform structure (an upward arching arc as opposed to a syncline). This feature appears to have formed as a result of uplift of a late Pleistocene fluvial surface by about 20 feet (6 meters). Assuming a continuous uplift of 6 meters during the last twenty thousand years, the estimated minimum rate of uplift rate is ~0.3 mm/yr. Uplift appears to have separated a once continuous drainage system into the southwestward-draining Uvas Creek and the northeastward-draining Llagas Creek. The presence of this antiform suggests that the uplifted area may be bounded by blind thrust faults that have been active for at least 50,000 years and possibly in the Holocene (the name given to the last 11,000 years of the Earth's history -- the time since the end of the last major glacial epoch, or "ice age."). A fault trace mapped by McLaughlin and others (2000a) about 700 feet southwest of Chesbro Dam may be a thrust fault that is involved in uplift of the "Uvas Antiform".

5.2 Coyote and Anderson Dams

Coyote and Anderson Dams are located near the western margin of the Diablo Range, one of the ranges in the Coast Ranges geomorphic province. The Diablo Range is about 130 miles (210 km) long and as much as 30 miles (50km) wide, extending from the Carquinez Straits to Coalinga, and bounded by San Joaquin Valley on the east and, in our area of interest, the Santa Clara Valley on the west. Large, en echelon anticlinal folds with cores of Franciscan Complex rocks and Cretaceous rocks draped over the Franciscan core characterize the range. Younger rocks occur in the synclinal folds. A large antiform with a Franciscan core extends from the Livermore Valley to Panoche Valley (east of Hollister), about 90 miles, and dominates the range opposite Coyote and Anderson Dams. Survey data (Norris and Webb, 1990) indicates that northeast of San Jose the range is being uplifted 1.5-2.0 mm/year.

A system of thrust and reverse faults sub-parallel to the strike-slip faults of the San Andreas system has been cited as evidence for a transpressive tectonic regime associated with the plate boundary (Coyle and Anderson, 1992). The term transpressive is used to describe the tectonic regime and suggests that movement involving components of both strike-slip and reverse movement has occurred due to the left-hand stepover of slip from the Calaveras to the Hayward fault in a right-hand strike-slip system (Wiegers and Tryhorn, 1992). In the Morgan Hill area, the Silver Creek fault, Coyote Creek fault, and Range Front Thrust have been mapped west of the Calaveras fault, the latter two along the eastern boundary of the Santa Clara Valley and the southwest-facing slope of the Diablo Range. Coyle and Anderson (1992) indicate the Silver Creek fault to be a steeply dipping, presumably reverse fault, and, from east to west, the Coyote Creek and Range Front Thrust to be thrust faults. At the Thomas Grade Road exposure of the Range Front Thrust, steeply, east-dipping Plio-Pleistocene sediments are faulted over old alluvium along a 15° east-dipping thrust fault (Coyle and Anderson, 1992).

5.3 Anderson Dam

The Coyote Creek fault is evident in the vicinity of Anderson Dam in the quarry upstream of the left abutment and is mapped on the right side of the spillway and spillway approach channel. This thrust fault cross the upstream right abutment dam foundation, including one trace marked by the contact between serpentinite and a suite of sandstone, shale, and greenstone rocks. Serpentinite is displaced over a basal unit of terrace deposits along a trace of the fault in an area of an over-steepened slope along the approach channel. The fault is truncated by terrace deposits and the latest age of faulting of 100,000 years was ascribed to this trace (Shlemon, 1983). The serpentinite occurs as boulder, cobble and smaller-size pieces in a matrix of crushed serpentinite.

5.4 Coyote Dam

The active trace of the Calaveras fault trends generally north-northwestward (upstream-downstream) near the left abutment of Coyote Dam. The Right Abutment fault, which strikes almost due north and is considered to be a splay off the Calaveras fault, passes through the right portion of the dam, the right abutment, the outlet pipe, and a portion of the spillway. GPS and terrestrial survey results indicate that new monitoring points Nos. 20 and 1 (most westerly) on the left abutment (west) side of the Calaveras fault are showing a very clear northwestward (N31°W) trend of movement at a rate of approximately 16.1 mm/yr. (0.34 ft. in 6.46 years). This rate of movement is in close agreement to the 15.5 mm/yr. rate of Coyote fault movement being recorded at Mendoza Ranch. This rate of fault movement, if assumed constant with time, suggests that the total fault movement in the left abutment since 1936 is on the order of 1027 mm (3.37 feet), compared to the maximum measured station/offset movement of between 1.1 and 1.2 feet along the dam crest. This is not a discrepancy, but rather reflects the apparent 40+ year time delay that it took for fault movement to propagate through the 60-ft thick embankment from the foundation to the dam crest. During this time period, the amount of tectonic related movement on the dam crest was very small. Recent GPS measurements indicate that the dam crest on the west side of the fault is moving at a rate of about 17 mm/yr, which is the same as other bedrock sites.

A series of high-angle, dip-slip fault traces, collectively referred to as the Right Abutment fault because they pass through the right abutment area of the dam, encompass a zone about 125 feet wide. The Right Abutment fault also passes through the foundation of the spillway and the new outlet tunnel. Earth Science Associates (ESA, 1989) calculated an average uplift rate of 0.2 mm/year for the Right Abutment fault based on elevation differences and regional Quaternary chronology. Although minimal, uplift and differential movement along the Right Abutment fault has the potential to adversely affect the spillway and outlet tunnel. The outlet tunnel was specifically designed and constructed with shorter sections and flexible joints to accommodate likely fault movement. The spillway floor has recently experienced cracking and spalling of the concrete floor that could be associated with Right Abutment fault movement.

According to Aydin and Page (1984), Coyote Reservoir occupies a pull-apart basin that has developed at a right-hand stepover in the trace of the Calaveras fault. A right step in a right-lateral shear system is a releasing or extensional geometry that enables the block between two en echelon fault strands to subside.

5.5 Guadalupe Dam

Bailey and Everhart (1964) mapped a relatively wide (up to about 2000 ft) northwest-trending fault zone they named the Ben Trovato fault zone along the southwest portion (left abutment) of the dam and southwestern edge of Guadalupe reservoir. The Ben Trovato fault zone is shown merging into the Shannon fault about 3.5 miles to the northwest. More recent mapping by McLaughlin and Helley (2000a) and McLaughlin and others (2000b) does not show the Ben Trovato fault zone. Instead, the more recent maps show an approximately 2-mile long fault trace passing through the long (northwest-trending) axis of the reservoir and beneath the central portion of the dam. The fault is shown as concealed (dotted) beneath the dam and reservoir and inferred (dashed) southeast of the reservoir. This fault trace may be an older Franciscan-type fault, or a subsidiary trace of the Berrocal fault. The Ben Trovato and Berrocal faults are not considered to be capable of producing significant earthquakes by the State of California (Petersen and others, 1996) or the Working Group on Northern California Earthquake Potential (WGNCEP, 1996). Currently, Guadalupe dam crest monument surveys do not indicate any obvious movement related to local tectonic movements.

5.6 Lenihan Dam

Lenihan Dam is located on the upper plate of the southwest-dipping Berrocal fault zone about 0.6 mile southwest of its main surface trace. This fault, together with the Monte Vista fault to the northwest and the Shannon fault to the southeast, forms a major tectonic element of the Santa Cruz Mountains.

McLaughin and others (2002) mapped a series of 2 to 3 sub-parallel fault splays along the east side of Lexington Reservoir, which they identified as the Lexington Fault Zone. As these faults extend southward they curve to the east, merging with the San Andreas fault zone at the upstream end of Lexington Reservoir. To the north the inferred western traces of the Lexington Fault Zone become lost in the highly sheared Franciscan Complex rocks. These traces could be affecting Lenihan Dam as evident by historic lateral movement of dam crest monuments.

Historic lateral movement as documented by crest monument surveys show that up until the 1989 Loma Prieta earthquake, the western 2/3 of the dam crest moved downstream (0.3 foot maximum) and the eastern 1/3 of the crest has moved upstream (0.2 foot maximum). The Loma Prieta earthquake created additional lateral movement similar in orientation to that recorded earlier amounting to 0.2 foot maximum downstream and 0.1 foot maximum downstream. The overall movement suggests right-lateral (or clockwise rotational) displacement of the embankment crest totaling a maximum of 0.6 foot on the western 2/3 and 0.2 foot on the eastern 1/3 from 1954 to 1999. This corresponds to maximum displacement rates of 4 mm/yr on the western 2/3 of the dam and 1.5 mm/yr on the eastern 1/3. Measured movements within the abutments are less than 0.2 foot (1.5mm/yr). Splays of the Lexington fault zone could underlie the dam and thus cause the surveyed monument velocities, which appear to be plausible tectonic rates. It is difficult, however, to explain the greater amounts of movement in the embankment relative to the abutments with faulting in the foundation rock. One possible explanation for the surveyed movements at Lenihan Dam is cross-valley compression (RLVA, 1999).

5.7 Calero Main Dam

Mapping of Calero Dam and vicinity by Bailey and Everhart in 1964 and McLaughlin and others (2001) indicates that an inferred fault trace, known as the Calero fault, underlies the right abutment of the main dam. As discussed in section 6.2 below, seepage in the right abutment area may be related to shearing of bedrock along a probable fault zone.

6.0 Seepage Concerns at District Dams

Seepage at the 11 District dams has never posed a major concern during their 65+ years since original construction. The only noted seepage that has posed a recurring maintenance problem for the district has been at Almaden, Calero and Uvas dams. These issues, and the geologic conditions surrounding them, are discussed below.

6.1 Almaden Dam

During construction of Almaden Dam the contractor encountered silica-carbonate in the left abutment and had to alter the alignment of the dam to accommodate the poor rock. As a result of the poor foundation rock, leakage problems (seepage) have been present since dam construction along the downstream left abutment and toe of Almaden Dam. With time, the leakage has provided water for vegetation growth that supports wildlife habitat that is now (January, 2003) causing environmental concerns as plans are being developed to collect and monitor the seepage.

The occurrence of silica-carbonate rock in the left abutment of Almaden Dam during excavation of the cut-off trench presented a significant challenge with respect to constructing the cut-off trench in a suitable foundation. The presence of the silica-carbonate material encountered in the left abutment side of the cut-off trench was perhaps the principal reason for placement of a fillet fill (not shown on the as-built drawings of the dam) over a portion of the downstream shell on the left abutment side of the dam following construction. Although very tough to penetrate, and notorious to drillers for grinding down drill bits, silica-carbonate is typically full of large voids due to removal and alteration of serpentinite minerals during hydrothermal alteration. Marliave (1936) states that "*the rock is badly crushed and honeycombed and is not uniform in composition or character. It may be soft or hard and contains irregular open cracks. It contains silica skeletons and carbonate segregations with much clay and numerous soft limonite pockets. It is treacherous rock in which to land a cut-off for a dam.*" Consequently, the original alignment of the cut-off trench had to be altered during construction so that it would terminate into more favorable serpentinite bedrock as discussed in Appendix B of this report.

6.2 Calero Dam

One of the main geologic issues at Calero Dam is seepage through the right abutment of the main dam, which has produced both perennial and ephemeral moist areas at or near ground surface long enough to have produced characteristic communities of hydrophytes or water-loving plants. Seepage through the main dam has been attributed to various adverse conditions related to the bedrock or embankment permeability. The pattern of seepage at Calero Dam suggests that reservoir seepage is flowing through the foundation and then onto the ground surface or into more permeable layers of the embankment.

Adverse bedrock characteristics include the presence of highly fractured rock and shear zones in the vicinity of the right abutment of the main dam. Wahler (1966) described the bedrock in the right abutment of the main dam as "an intensely fractured, intrusive diabase which is in contact with the shear zone and forms most of the right abutment of the dam. This rock possesses relatively high secondary permeability because of its fractured nature". A geologic cross section through the main dam and reservoir rim to the east was prepared by District geologist Jeremy Wire (1966), based on subsurface information from Wahler's 1966 report. The cross section shows the shear zone encountered in DH-1 as clay gouge with an inferred trace of the Calero fault zone dipping steeply eastward through the clay gouge. The fault zone is shown projected through the groin of the right abutment of the dam at its intersection with the foundation at about elevation 460 feet (NGVD 1929) (Wire, 1966).

6.3 Uvas Dam

Since its construction in 1956 and 1957 by the Gavilan Water District, Uvas Dam has had recurring seepage problems. Several attempts have been made to mitigate and control seepage flow through the embankment, foundation, and abutments. These efforts have included installation of several drains, grout curtains, and 6-inch diameter relief wells (RW-1 through RW-4). The relief wells are now used for monitoring piezometric levels.

The dam was constructed on Jurassic-Cretaceous Franciscan Complex rocks. The rock types near the dam include basalt, greenstone, limestone, chert, shale, and sandstone. The bedrock underneath the dam is blanketed with Quaternary deposits, which have been interpreted as consisting of artificial fill, colluvium, channel, terrace, and landslide deposits. During construction of the dam, a grout curtain was installed along the centerline of the cutoff trench. In May 1958, a second grout curtain, consisting of 18 grout holes, was placed between the spillway weir and the left end of the dam. In 1996, the District installed a double-row grout curtain in the foundation bedrock of the right abutment. This later effort appears to have significant reduced the seepage, especially that emanating through the right abutment.

7.0 Seismic Considerations

Following the great San Francisco Earthquake of 1906, there was a relatively long span of seismic quiescence in the San Francisco Bay Area. We now understand that this is a common phenomenon, especially following large earthquakes (M>7), wherein after major strain energy has been released due to the earthquake, the fault remains relatively dormant while the strain energy begins to build up once again for the next characteristic event. This delay time is referred to as the recurrence interval and is unique for each fault based on its seismotectonic setting. For the San Andreas fault in the San Francisco Bay Area, the recurrence interval for characteristic events is about 210 years (Working Group on Northern California Earthquake Potential, 1996). It wasn't until the 1984 Morgan Hill event, which occurred on the Calaveras fault with its epicenter in southeast San Jose near Halls Valley, and produced significant ground shaking at Anderson and Coyote dams, that any of the 11 District dams were impacted by strong ground shaking since their construction in the 1930s, 1940s, or 1950s. This appendix presents a discussion of the performance history of several District dams during the 1984 M_L 6.4 Morgan Hill and 1989 M_L 6.9 Loma Prieta Earthquakes. Using current earthquake attenuation theory, estimates are also provided for the estimated peak ground acceleration that would occur at each District dam in the event the maximum expected earthquake were to occur on any of the major earthquake faults located within close proximity to the dams. Using an approximate method, estimates are also provided for earthquake-induced displacements for major earthquakes that have an estimated high probability of occurrence in the next 30 years.

7.1 General Earthquake Information

Earthquake scientists consider it a very high probability (combined probability of 67%, Working Group on California Earthquake Probability or WGCEP, 1999) that within the next 30 years a large damaging earthquake will occur within the Bay Area on either the Peninsula segment of the San Andreas, Hayward, or Calaveras faults similar to the Loma Prieta earthquake. This is shown on Figure C-1. While the Loma Prieta earthquake of October 17, 1989, produced relatively high accelerations, its duration of shaking was relatively short (less than 15 seconds). During the next significant event, scientists expect the duration to be longer.

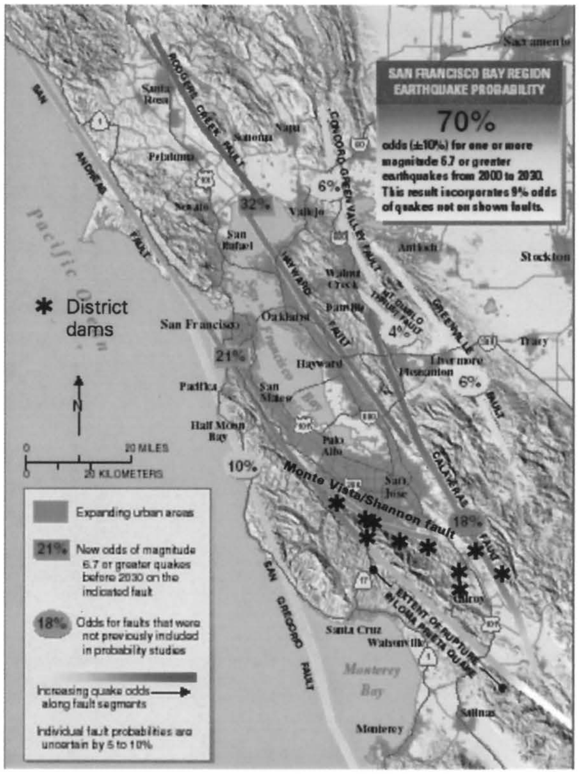

SAN FRANCISCO BAY REGION EARTHQUAKE PROBABILITY
70%
odds (±10%) for one or more magnitude 6.7 or greater earthquakes from 2000 to 2030. This result incorporates 9% odds of quakes not on shown faults.
SAN ANDREAS FAULT
RODGERS CREEK FAULT
CONCORD-GREEN VALLEY FAULT
HAYWARD FAULT
MT. DIABLO THRUST FAULT
GREENVILLE FAULT
CALAVERAS FAULT
SAN GREGORIO FAULT
Monte Vista/Shannon fault
EXTENT OF RUPTURE IN LOMA PRIETA QUAKE
32%
6%
4%
6%
21%
10%
18%
Pacific Ocean
Sacramento
Santa Rosa
Napa
Sonoma
Petaluma
Novato
Vallejo
San Rafael
Antioch
Stockton
Walnut Creek
Danville
Oakland
San Francisco
Tracy
Livermore
Pleasanton
Hayward
Pacifica
San Mateo
Half Moon Bay
Palo Alto
San Jose
Gilroy
Santa Cruz
Watsonville
Monterey Bay
Monterey
Salinas
✱ District dams
N
0
20 MILES
20 KILOMETERS
Expanding urban areas
21% New odds of magnitude 6.7 or greater quakes before 2030 on the indicated fault
18% Odds for faults that were not previously included in probability studies
Increasing quake odds along fault segments
Individual fault probabilities are uncertain by 5 to 10%

Figure C-1. Threat of major earthquakes before 2030 and District Dams

(with District dams shown by asterisks)

(estimated at about 25-30 seconds), and thus potentially more damaging, than what was experienced during the Loma Prieta earthquake. For a magnitude 8+ event occurring on the San Andreas fault (the Calaveras, Hayward, and Shannon-Monte Vista faults are not considered capable of such a great earthquake), one can expect the duration to be about 45-60 seconds or longer. All District facilities are located sufficiently close to one or more of these faults that they will likely experience future significant seismic shaking. Several District dams could deform and settle 2 to 3 feet during a significant earthquake (8+ magnitude) on the San Andreas. It should be noted that a number of District dams, including Lenihan, Stevens Creek, Guadalupe, Anderson, and Calero have had detailed dynamic finite element method analyses performed under simulated design earthquake scenarios. Other dams, including Coyote, Anderson, Uvas, Vasona, and Rinconada Reservoir have had detailed stability analyses performed under simulated earthquake loading. All District dams have been judged safe under major earthquake shaking except for Stevens Creek dam which underwent major construction to significantly improve its safety under future seismic loading, as discussed later in Section 7.5.

7.1.1 Primary Waves

There are two distinct types of energy radiation or propagation that emanate from an earthquake. Such energy release is customarily referred to as earthquake energy wave propagation or simply earthquake waves. The two types of waves consist of *body waves* that propagate through the body of the earth, and *surface waves* that develop as body waves exit the earth's surface. These waves are depicted graphically in Figure C-2. Body waves spread out spherically from the point of the earthquake fault rupture. The *epicenter* is defined by the point on the earth's surface normal to the center of the rupture zone. The *focal depth* is the depth from the epicenter to the start of the earthquake fault rupture. Body waves travel significantly faster than surface waves. The body wave that travels the fastest is called the primary wave, or P wave. P waves arrive at a given point before any other type of seismic wave. P waves travel through solids, liquids and gases. They move through the earth at different speeds, depending on the density of the material through which they are moving. As they move deeper into the earth, where material is denser, they speed up. The average velocity of P waves for rocks is about 6.1 km/sec (3.8 mi/sec).

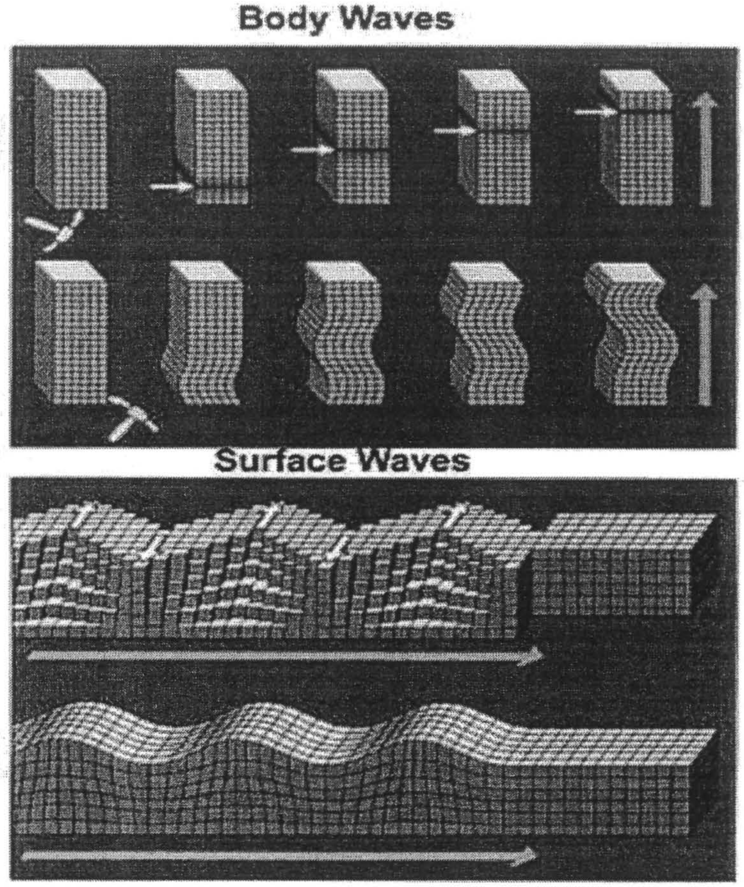
P waves are push-pull waves. As P waves travel, they push rock particles into the particles ahead of them, thus compressing the particles. The rock particles then bounce back. They hit the particles behind them that are being pushed forward. The particles move back and forth in the direction the waves are moving.

7.1.2 Secondary Waves

The body wave that travels through the earth about one-half as fast as P wave is referred to as secondary wave, or S wave. S waves arrive at a given point after P waves do. S waves travel through solids but not through liquids and gases. Like P waves, S waves speed up when they pass through more dense material. S waves cause rock particles to move from side to side like a shearing action. The rock particles move at right angles to the direction of the waves. The average velocity of S waves for rocks is about 3.5 km/sec (2.2 mi/sec).

7.1.3 Surface Waves

The slowest-moving seismic waves are called surface waves, and are composed of Rayleigh (R wave) or Love (L waves). L waves arrive at a given point after primary and secondary waves do. L waves originate at the earthquake epicenter. Surface waves travel along the surface of the earth, rather than down into the earth. Although they are the slowest of all the earthquake waves (R slower than L which is about 0.9 times the S wave, or about 2 mi/sec), L waves usually cause more damage than P or S waves. A simplified diagram showing the types of particle movement for the different types of body and surface waves is presented in Figure C-2 below.



Body Waves

Surface Waves

Figure C-2 Types of Body and Surface Waves Generated by Earthquakes

7.2 Response of Embankments to Earthquakes

Some seismic waves have more potential to cause damage to dams than others. Typically, seismic waves will amplify the seismic motion when they move upward from the bedrock into the dam. The relative motion at the top of the dam is amplified between about 2-3 times the motion in the bedrock for earthquakes great than about magnitude 6 or greater (Harder et al., 1990).

The predominant period of earthquake motion (largest motion) will be in the center of the dam where its vertical thickness is greatest, relative to the abutments where the embankment is thin.

The difference in the period of fundamental shaking between the center of dam and the hinge points at the abutment produces larger displacements as one goes from the abutments to the center of the dam. Another factor affecting frequency is the contrast between the higher strength of the bedrock at the abutments and the relatively weaker earth materials of the embankment. The relative response in motion between the center (embankment) and the abutments (bedrock) produces transverse cracking (perpendicular to the dam crest).

As mentioned previously, peak ground acceleration values at any dam vary depending on the earthquake magnitude and distance of the dam from the causative fault. Numerous researchers have developed mathematical relationships for the decay (attenuation) of earthquake acceleration as a function of magnitude and distance. The relationship considered most appropriate for District facilities was developed by Abrahamson & Silva (1997). Using this relationship, the estimated peak ground (bedrock) acceleration was computed for various magnitude faults as a function of distance and is presented in Figure C-3. As shown in Figure C-3, peak bedrock acceleration values were computed for earthquake magnitudes of 7.0, 6.5, 6.0 and 5.0 for distances up to 50 kilometers from the site. As shown in Figure C-3, it is obvious that the greatest percentage of attenuation occurs within the first 15 km from the earthquake epicenter.

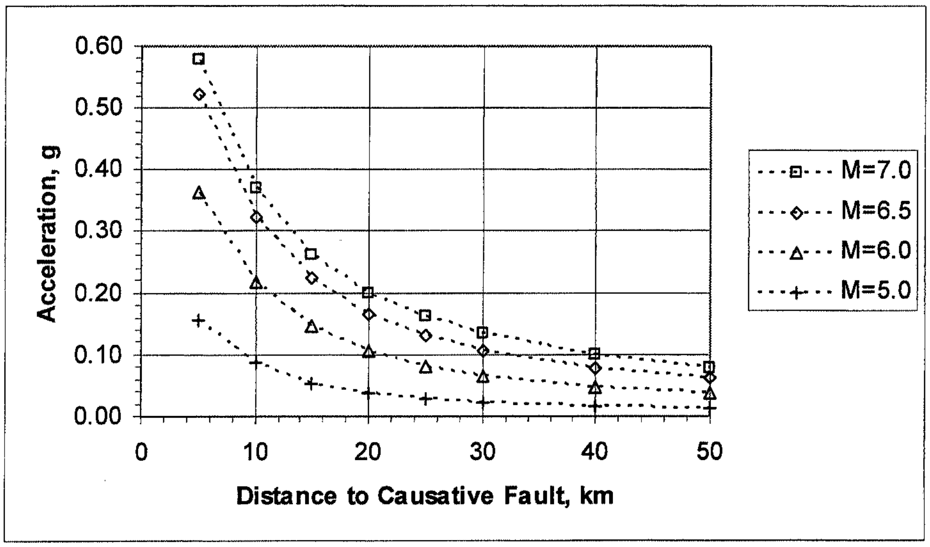


Figure C-3. Attenuation of earthquake acceleration as a function of distance to causative fault

Another interesting relationship is presented in Figure C-4. This figure shows the earthquake magnitude causing 0.1 g peak ground acceleration as a function of distance from the causative fault. The significance of this relationship is that structures would experience the same acceleration (namely 0.1 g) if they were situated 5 km from a magnitude 4.5 earthquake or 40 km from a magnitude 7.0 earthquake.

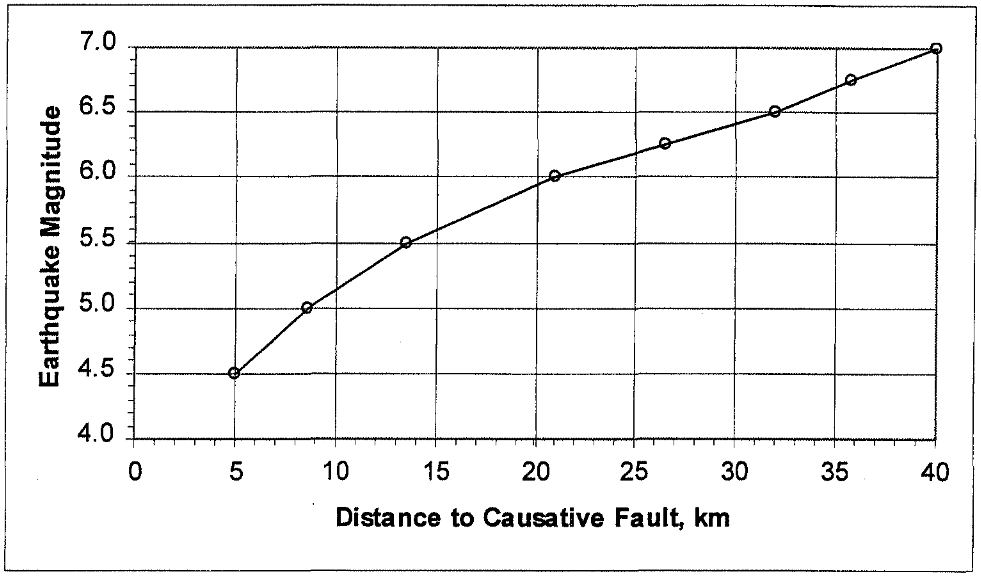


Figure C-4. Earthquake acceleration of 0.1g as a function of distance to causative fault and earthquake magnitude

7.3 The Actively Creeping Calaveras Fault and Coyote Dam

Of all District dams, Coyote Dam is the only one knowingly constructed over an active fault. The dam was designed and constructed to withstand a total of about 20 feet of expected movement along the Calaveras fault during a large seismic event. This amount of offset was estimated by engineers and geologists based on the movement caused elsewhere by the 1906 San Francisco Earthquake. Unique design and construction was conceived to prevent internal erosion in case of fault offset in the foundation. The core of the dam was constructed of impervious clay with an extra wide crest. A layer of gravel was placed on the upstream and downstream faces of the clay core. The purpose of the gravel is to fill in any cracks formed during fault movement. The gravel was in turn overlain with loose rock. The weight of the rock was designed to push the gravel into any cracks or openings as they form. As indicated by historic plots of monument survey data, the dam has experienced right lateral movement as a result of ongoing active creep on the Calaveras fault.

7.4 High Pore Pressures and Earthquake-Induced Seismic Affects at Lenihan Dam

Findings of recent geotechnical investigations of the outlet pipe and Lenihan Dam (RLVA, 1999) indicate that high pore pressures surround the outlet pipe and the upper portion of the embankment. Possible contributing factors include the unique characteristics of the dam, specifically the highly impermeable nature of the clay core, and intense shaking the dam experienced during the Loma Prieta earthquake. Because high pore pressures are present, instrumentation at Lenihan Dam includes an accelerometer. Earthquakes that produce seismic accelerations greater than 0.05g will be detected by the accelerometer, which will in turn trigger a computer to collect more frequent readings of the vibrating wire piezometers near the outlet pipe and in the dam. Dynamic analysis (mathematical model) was performed using the design earthquake (magnitude 8±) along the San Andreas fault. Results indicate that there will not likely be significant failure of Lenihan Dam. However, during full reservoir combined with a significant earthquake, 2 to 3 feet settlement of the dam is expected. The dam has been instrumented with two strings of bi-axial tiltmeters connected to an automated recording system in order to determine any future earthquake-induced deformation of the dam.

7.5 Seismic Retrofit of Stevens Creek Dam

Of all District dams originally analyzed during the late-1970s and early-1980s for earthquake performance, Stevens Creek Dam is the only one found to be deficient with respect to significant seismic shaking. The District's consultant, Wahler and Associates, determined that the upstream impervious section of the dam would not withstand the maximum credible earthquake, resulting in failure of the upstream section and significant loss of freeboard. Although there would be significant damage to the dam during such an earthquake, the dam was not expected to fail catastrophically. In 1985, in order to provide seismic safety of Stevens Creek Dam, new buttress fill sections were constructed on the dam. A buttress fill was placed over the upstream toe of the dam to stabilize the upstream slope, and the dam crest was raised by placing new fill on the downstream face to provide adequate freeboard.

7.6 Response of Dams to the Morgan Hill and Loma Prieta Earthquakes

This section discusses specific impact to District Dams in response to the Morgan Hill and Loma Prieta Earthquakes.

A summary of the types of disturbance and relative movements (vertical and horizontal) of District dams affected by the Morgan Hill earthquake is presented on Table C-1. A summary of the types of disturbance and relative movements (vertical and horizontal) of District dams affected by the Loma Prieta earthquake is presented on Table C-2.

Table C-1. Affects of April 24, 1984 Morgan Hill Earthquake on District Dams

District Dam	Location of Disturbance	Type of Disturbance	Dimensions of Disturbance	Vertical Settlement	Lateral Movement
Anderson	Along crest corresponding to core/shell contacts	Longitudinal cracking	Two linear sets 20 ft apart – 920-1100 ft long 2 to 6.5 ft deep	0.6 in	0.4 in
Chesbro	---	---	---	0.6 in	---
Coyote	U/S face, crest, spillway	Minor cracking	Largest cracks 10 ft to 90 ft long - 30 in deep	2.6 in	1.4 in
Uvas	---	---	---	1.0 in	---

Notes:
Abbreviations: in. = inches; ft. = feet; U/S = upstream, D/S = downstream, --- = not applicable

Table C-2. Affects of October 17, 1989 Loma Prieta Earthquake on District Dams & Austrian Dam

District Dam	Location of Disturbance	Type of Disturbance	Dimensions of Disturbance	Other minor disturbance	Vertical Settlement	Lateral Movement
Almaden	None	---	---	---	1.0 in	0.4 in
Anderson	Along crest corresponding to core/shell contacts	Longitudinal cracking	Up to about ¾ in. wide	---	1.6 in	1.0 in
Austrian +	1) Both abutments 2) U/S & D/S faces	1) Transverse cracking 2) Longitudinal cracking	1) 8 in to 1.5 feet wide, 10 ft to 27 ft deep @ lft crest	---	33.6 in	18 in
Calero	None	----	----	---	1.3 in	0.8 in
Chesbro	Central portion of upstream edge of crest	Longitudinal cracking *(beginning of slump landslide)	240 ft. long, 4 in. wide, 4 in. vertical offset	Cracks in right upstream groin	2 in	0.6 in
Coyote	None	---	---	---	---	---
Guadalupe	1) Near left and right abutment 2) along crest of U/S berm	1) Transverse cracking 2) Longitudinal cracking	1) To depths of about 3 ft. 2) entire width of berm	Cracks in upstream concrete facing	7.7 in	1.8 in
Lenihan	1) U/S and D/S right abutment in embank. & native bedrock; D/S left abutment 2) upstream face	1) Transverse cracking 2) Scattered longitudinal	---	---	10.2 in	6 in. (2.4 in U/S @ rt. abut. and 3.6 in D/S @ lft abut.)
Rinconada	None	---	---	---	0.4 in	0.2 in
Stevens Creek	None	---	---	---	0.6 in	0.2 in
Uvas	None	---	---	---	0.7 in	None
Vasona	Crest of right embank. Section	**Longitudinal cracking	---	---	1.8 in	1.1 in

Notes:

Abbreviations: in. = inches; ft. = feet; U/S = upstream, D/S = downstream, --- = not applicable, embank. = embankment; rt. = right, lft = left, abut. = abutment

*if the duration of the Loma Prieta earthquake had been longer, the longitudinal cracking would probably have developed into a landslide.

**Longitudinal cracking along the embankment crest had previously been observed during the August 8, 1989 earthquake (M=5.1), a probable foreshock of the Loma Prieta earthquake. New longitudinal cracking was observed at the same locations, which appears to represent reactivation and possible enlargement of the previous cracking.

+ = Austrian Dam owned by San Jose Water Company

As shown on Table C-1, District dams that exhibited noticeable vertical settlements of about 0.6 inches to 2.6 inches due to the Morgan Hill earthquake, consisting in order of deceasing impact Coyote, Uvas, Chesbro, and Anderson Dams.

As shown on Table C-2, District dams that exhibited noticeable disturbance also experienced vertical settlements of up to 10.2 inches due to the Loma Prieta earthquake, consisting in order of deceasing impact Lenihan, Guadalupe, Chesbro, and Vasona dams.

7.6.1 April 24, 1984 Morgan Hill Earthquake

The Morgan Hill earthquake with a magnitude of 6.2 occurred on the southern segment of the Calaveras fault on April 24, 1984 near Halls Valley at the base of Mt. Hamilton northeast of San Jose, California. Earthquake related damage as a result of the Morgan Hill Earthquake occurred only at Leroy Anderson Dam and Coyote Dam Due to their proximity to the seismic source.

Leroy Anderson Dam is located about 10 miles from the epicenter, situated near Halls Valley. The following was taken from Tepel (1984) and Bureau and others (1989). Earthquake effects consisted of two linear sets of longitudinal cracks spaced approximately 20 feet apart in the asphaltic pavement along the crest. The crack sets were on opposite sides of the crest centerline and appeared to coincide with the buried upstream and downstream shoulders of the clay core. The upstream shoulder crack set had a linear extent of 1100 feet, while the downstream shoulder crack set extended about 920 feet. As revealed by nine backhoe trenches, the downward extent of the crack sets were minor with general maximum depths of 3-1/4 feet for the upstream crack set and 6-1/2 feet for the upstream crack set. The maximum surface openings of ¾ inch coincided with the thicker sections of the dam, with crack width decreasing to hairline near the abutments. Formation of the cracking over the buried shoulder of the core appeared to be related to the contrast in seismic response between the clay core and outer rockfill shells, and to preferential settlement of the rockfill shells relative to the core.

Coyote Dam is located about 15 miles from the epicenter. The following was taken from Bureau and others (1989). Damage to the dam was minor and consisted of a discontinuous series of small cracks in reservoir sediment and slope wash deposits between large boulders on the upstream face, and associated large cracks which indicate possible dislocation of small groups of these boulders. The cracks were generally close to the maximum section of the dam and were seen from slightly below water line to ~el. 772 to ~el. 778 feet. The cracks in the infilling sediment were generally open up to 1 to 2 inches and a few feet long. The cracks affecting groups of boulders were open from several inches to about 1 foot wide, and were up to about 10 feet long. The maximum depth determined by probing was on the order of 5 to 7 feet. All of these cracks seemed to be the result of rearrangement of the boulders as a result of the earthquake. Only minor cracks were observed in the spillway access road near the right abutment and on the crest near the left abutment contact. During the Morgan Hill Earthquake of April 24, 1984, Coyote Dam experienced some of the highest seismic accelerations ever measured (1.29 g).

7.6.2 October 17, 1989 Loma Prieta Earthquake

The Loma Prieta earthquake was unique because it produced significantly higher vertical accelerations than previously associated with strike-slip type faulting of comparable size

(~7.0 magnitude) (characteristic of the Santa Cruz Mountain segment of the San Andreas fault). Also, this earthquake had a much shorter duration than expected (~15 seconds or about half of the expected duration) for a magnitude 7.0 event. Generally speaking, peak accelerations were 0.5 to 0.6 times the force of gravity (1.0 g is the acceleration due to the force of gravity or 32.2 ft./sec^2) at District dams located near the Loma Prieta earthquake (Lenihan, Stevens Creek), and vertical accelerations were 1/3 to 2/3 of horizontal accelerations. The high vertical accelerations and short duration of the earthquake may be due to a bifurcation (splitting of the motion into two directions) of the fault rupture, with propagation of the rupture from the central portion of the fault segment toward the ends of the fault. Scientists generally expect the motions produced by most earthquakes as propagating from one end of the rupture zone to the other end.

The hypocenter of the Loma Prieta earthquake of October 17, 1989, was at a depth of 17 km on the southwest side of the San Andreas fault trace to a depth of ~8 km beneath the surface trace of the San Andreas fault (U.S.G.S. staff, 1990; Burgmann et al., 1997; Zoback et al., in press); the earthquake had a surface wave magnitude of 7.1 and a moment magnitude of 6.9 (McNutt and Toppozada, 1990). The earthquake is believed to have occurred along a fault segment of the San Andreas fault known as the Santa Cruz Mountain segment. Above the 8 km depth, slip on the terminated fault rupture is believe to have been taken up by uplift and shortening in the overlying crust, accommodated at ground surface by extensional fissuring, ridge-top spreading, and massive rotational slope failures along the southwestern side of the San Andreas fault (Ponti and Wells, 1991; McLaughlin and Clark, in press).

Initial fault rupture extended along a plane striking N50^{O}±10^{O}W and dipping 70 ^{O}±15SW. To account for this rupture geometry, which does not correspond to previous models of the geometry of the San Andreas fault, the following explanations have been proposed. 1) The San Andreas fault dips steeply southwestward at depth (U.S.G.S. Staff, 1990); 2) A SW-dipping reverse fault cuts a vertical San Andreas; and 3) Two separate dextral-reverse faults lie northeast and southwest of the San Andreas fault trace (Zoback, et al., in press; Prentice and Schwartz, 1991; McLaughin and Clark, in press; Burgmann et al. 1997). Movement was right-oblique, amounting to 6.2 feet along strike and 4.3 feet along dip. Coseismic deformation following the Loma Prieta earthquake affected a large region in the southern Santa Cruz Mountains. Maximum uplift at the ground surface amounted to more than 1.8 feet in the area 6 miles northwest of the epicenter and maximum downdrop was measured at more than 0.3 feet in the Santa Clara Valley.

The following sections summarize impacts of the Loma Prieta earthquake on District Dams.

Almaden Dam: There was no significant damage.

Leroy Anderson Dam: Extensive but minor cracking occurred along the alignment of crest guard rail posts. Minor longitudinal cracking also occurred along the crest in the same general areas that experienced cracking during the Morgan Hill earthquake. The largest cracks were up to ¾ inch wide and appeared to have formed where compacted fill had been placed to raise and extend the crest a few feet as part of the 1985 spillway enlargement project.

Guadalupe Dam: Some damage occurred to the upstream concrete facing, but was determined to be inconsequential. Damage consisted of downstream movement of several slabs a few inches, slightly overlapping slabs below. Transverse cracking was observed near the left and right abutments up to depth of approximately 3 feet. Scattered minor cracks were present on the crest. Longitudinal cracks were observed on the crest of the upstream berm suggesting movement of the lower portion of the upstream shell. The cracks were presents over virtually the entire width of the dam. The cracks were excavated and recompacted.

Calero Main Dam, Calero Auxiliary Dam, and Fellows Ranch dike: There was no significant damage to these structures.

Chesbro Dam: Cracks occurred predominantly in the right upstream groin with minor cracks along the upstream edge of the crest in the central portion of the dam. In the central portion the longitudinal cracking was about 240 feet long, 4 inches wide, with a vertical offset of 4 inches. The cracking that occurred in the central portion of Chesbro Dam was related to initiation of a small slump-type landslide. It the duration of the Loma Prieta earthquake had been longer, this feature would probably have developed into a landslide.

Coyote Dam: No cracking was observed.

Lenihan Dam: Transverse cracking occurred in both upstream and downstream directions at the right abutment with cracking localized to native ground immediately right of the abutment. Cracking in the left abutment was confined to the downstream groin. Scattered minor longitudinal cracks developed on the upstream face. The crest settled about 10.2 inches at its maximum section and moved about 7 inches downstream as a result of seismic shaking.

Although Lenihan Dam performed well during the Loma Prieta earthquake, Austrian Dam at Lake Elsman, located upstream of Lenihan, was extensively damaged. Because the dam did not retain any significant amount of water at the time of the earthquake, there was no damage to property or lose of life.

Rinconada Treated Reservoir and Dam: No damage or cracking reported.

Stevens Creek Dam: No damage or cracking reported.

Uvas Dam: No damage or cracking reported.

Vasona Dam: An extensive, but minor, longitudinal crack system developed at the crest of the right embankment section. Longitudinal cracking on the embankment crest had previously been observed during the August 8, 1989 earthquake (M=5.1) a probable foreshock of the Loma Prieta earthquake. New longitudinal cracking was observed at the same locations, which appears to represent reactivation and possible enlargement of the previous cracking.

7.7 Preliminary Assessment of Dam Vulnerability from Predicted Earthquakes

Figure C-1 shows the location of significant faults with the potential for earthquakes that could affect District dams. The analysis in this section attempts to provide a predictive tool to assess relative vulnerability of District Dams so that future response to particular seismic events can be planned. The vulnerability of each dam was assessed for future earthquakes (with estimated magnitudes in parenthesis) on the San Andreas Peninsula fault segment (7.1), the entire length of the Calaveras fault (7.0), the Southern Hayward fault segment (6.9), and the Monte Vista/Shannon fault (6.8).

As mentioned in Section 7.0, the WGCEP (1999) identified several faults with the probability of producing magnitude 6.7 or greater earthquakes in the San Francisco Bay Area in the next 30 years (see Figure C-1). For our analysis we only consider those seismic events that would have the greatest potential to adversely affect District dams. In their report they state (WGCEP, 1999):

> *"The Hayward–Rodgers Creek, San Andreas, and Calaveras Fault systems have the highest probabilities of generating a* $M \geq 6.7$ *earthquake before 2030. These faults pose a direct threat to the cities of San Francisco, Oakland, and San Jose, which ring San Francisco Bay. The Hayward Fault is of particular concern because of the density of urban development along it and the major infrastructure lines (water, electricity, gas, transportation) that cross it."*

As mentioned previously, the calculated probability of occurrence of a magnitude 6.7 in the next thirty years (2000 to 2030) is 32 % for combined fault slip on the Rodgers Creek and Hayward faults, 21 % on the San Andreas Peninsula fault segment, and 18 % on the entire Calaveras fault segment. While the above analysis indicates the probability of magnitude 6.7 or greater earthquakes, the WGCEP study (1999) does not provide specific likely earthquake magnitudes for each seismic event. Consequently, for our vulnerability analysis we needed to decide what earthquake magnitudes would be appropriate for each of the above-mentioned faults or fault segments. The Working Group on Northern California Earthquake Potential (WGNCEP, 1996) does, however, provide the following likely earthquake magnitudes: 1) a magnitude 6.9 for the Southern Hayward fault segment with an effective recurrence time of 210 years; 2) a magnitude 7.1 for the Peninsula San Andreas fault segment with an effective recurrence time of 400 years; and 3) a magnitude 7.0 for the entire Calaveras fault with an effective recurrence time of 400 years.

In addition, we analyzed the potential adverse affects on District dams due to a 6.8 earthquake on the Monte Vista/Shannon fault system. This fault system was not identified by the WGCEP (1999) as capable of producing an earthquake in the next 30 years. However, because the Monte Vista/Shannon fault is 1) very close to several District dams, 2) consists of a range-front thrust, and 3) is capable of generating a 6.8 earthquake (WGNCEP, 1996), we felt it would be prudent to analyze the vulnerability of District dams to such an earthquake.

7.7.1 Preliminary Seismic Vulnerability Analysis of District Dams

One of the most significant hazards to safety of District dams is intense seismic loading from future earthquakes. As a dam owner, the District needs to assess the relative vulnerability to future seismic events. This section describes a preliminary assessment to attempt to semi-quantify seismic vulnerability based a correlation developed by Bureau and others (1985) between Relative Vertical settlement and Earthquake Severity Index.

Our analysis of vulnerability focused on attempting to develop a correlation similar to that of Bureau and others (1985) using reported vertical settlements of District dams and other local dams from the two most significant seismic events since dam construction, namely the Loma Prieta and Morgan Hill Earthquakes, as described in detail above in Section 7.6. For our analysis we also used estimated settlements from seismic analysis on Anderson Dam by W.A. Wahler & Associates (1971) and on Guadalupe, Almaden, and Calero Dams by Wahler Associates (1981).

The Loma Prieta earthquake provided the most significant test of dam performance during the 50- to 65-year existence of District dams. Overall, the dams performed more than adequately on October 17, 1989, with surprisingly minimal threat to public safety. Chance circumstances on October 17, however, may have contributed to favorable performance such as a relatively short duration of shaking and the nearly empty reservoirs from the antecedent drought conditions. Because the Loma Prieta earthquake did not provide a near worst-case scenario, dam owners should not be lulled into thinking a future comparable seismic event would also result in adequate dam performance. Consequently, diligence would dictate that the dams be assessed for general vulnerability to future seismic events, specifically those predicted to occur using probabilistic methods in the next 30 years in the Bay Area by the WGCEP (1999).

As mentioned above, the WGCEP (1999) assessed the likelihood that several seismic events of magnitude 6.7 or greater, comparable to the Loma Prieta earthquake, would occur on major Bay Area faults in the Bay Area with varying probabilities in the next 30 years (2030). Consequently, we can expect District dams to respond similarly to these predicted earthquakes. Obviously, distance of the dam to the causative fault governs the dam's relative response. Seismologists have determined that this response attenuates rapidly (logarithmically) with distance.

To attempt to predict dam disturbance as the result of future seismic loading from likely significant earthquakes in the Bay Area, we used a correlation and methodology originally formulated for concrete face rockfill dams developed by Bureau and others (1985) that relates Relative Settlement to Earthquake Severity Index. Relative Settlement is defined as the ratio between change in height (vertical displacement) due to earthquake shaking in millimeters and the height of the dam in meters producing the ratio mm/m in per thousands. Earthquake Severity Index (ESI) is the product $A*(M-4.5)^3$; where A is acceleration g and M is the earthquake magnitude. Table C-3 summarizes pertinent Relative Settlement and ESI data from Bureau and others (1985) and data from District Dams and Austrian Dam, which is owned by the San Jose Water Company.

Figure C-5 shows a plot of the published data from Table C-3 for various rockfill dams situated around the world. The data in Figure C-5 include both observed vertical settlement during actual earthquakes and computed settlement.

Table C-3. Relative Settlement and ESI Data

Dam Name (location)	H (m)	Delta H (mm)	Relative Settlement mm/m	PGA (g)	M	ESI	Reference
Cogoti (Chile)	84	381	4.54	0.20	8.3	11.0	Bureau and others (1985)
Miboro (Japan)	131	30	0.23	0.20	7.0	3.1	
Minase (Japan)	67	61	0.91	0.08	7.5	2.2	
Oroville (Calif.)	235	9	0.04	0.10	5.7	0.2	
El Infiernillo (Mex.)	148	130	0.88	0.12	7.6	3.6	
La Villita (Mex.)	60	45	0.75	0.10	7.6	3.0	
Leroy Anderson (Calif.)	72	15	0.21	0.41	6.2	2.0	
Terror Lake (AK)	50	600	12.00	0.35	8.5	22.4	
Terror Lake (AK)	50	300	4.00	0.50	6.5	4.0	
Cirrata (Indonesia)	125	300	2.40	0.35	6.5	2.8	
Publo Viejo (Guatemala)	134	4450	33.33	0.65	8.3	34.3	
Chicoasen (Mexico)	240	1000	4.17	0.85	7.0	13.3	
Fortuna (Panama	104	800	7.69	0.40	7.5	10.8	
USBR (Example Dam)	213	700	2.72	0.43	6.5	3.5	
Dames & Moore (Ex. Dam)	100	487	4.87	0.70	7.1	12.3	
Almaden	33	24	0.71	0.37*	6.9	5.4	M6.9 Loma Prieta Eq. Tepel & others (1996) Harder & others (1998)
Anderson	73	38	0.52	0.18	6.9	2.5	
Calero	30	32	1.09	0.29	6.9	4.0	
Coyote	42	0	0.00	0.17	6.9	2.5	
Chesbro	29	50	1.73	0.29	6.9	4.0	
Guadalupe	39	188	4.78	0.37	6.9	5.4	
Lenihan	59	250	4.20	0.66	6.9	9.4	
Rinconada	12	9	0.72	0.37	6.9	5.4	
Stevens Creek	39	15	0.37	0.26	6.9	3.5	
Uvas	32	18	0.55	0.37	6.9	5.4	
Vasona	9	44	4.82	0.30	6.9	5.4	
Austrian +	56	735	13.03	0.77	6.9	10.6	
Anderson	73	15	0.21	0.57	6.2	2.8	M6.2 Morgan Hill Eq. Tepel & others (1996)
Coyote	42	67	1.59	0.61	6.2	3.0	
Chesbro	29	15	0.53	0.26	6.2	1.3	
Uvas	32	24	0.76	0.20	6.2	0.7	
Almaden	33	610	18.51	0.59	8.5	37.9	M.C.E. 8.5 San Andreas Wahler (1976, 1981)
Calero	30	610	20.61	0.45	8.5	28.7	
Guadalupe	39	2225	56.57	0.59	8.5	37.9	
Lenihan	59	1524	25.63	0.89	8.5	56.8	

Notes:

1) * = shaded values of PGA calculated using Attenuation relationship of Abrahamson & Silva (1997)
2) + = Austrian Dam owned by San Jose Water Company
3) PGA = peak ground acceleration; M = earthquake magnitude; ESI = Earthquake Severity Index (see text)
4) H = height of dam in meters; Delta H = change in height due to earthquake, Relative Settlement = Delta H/H (see text)

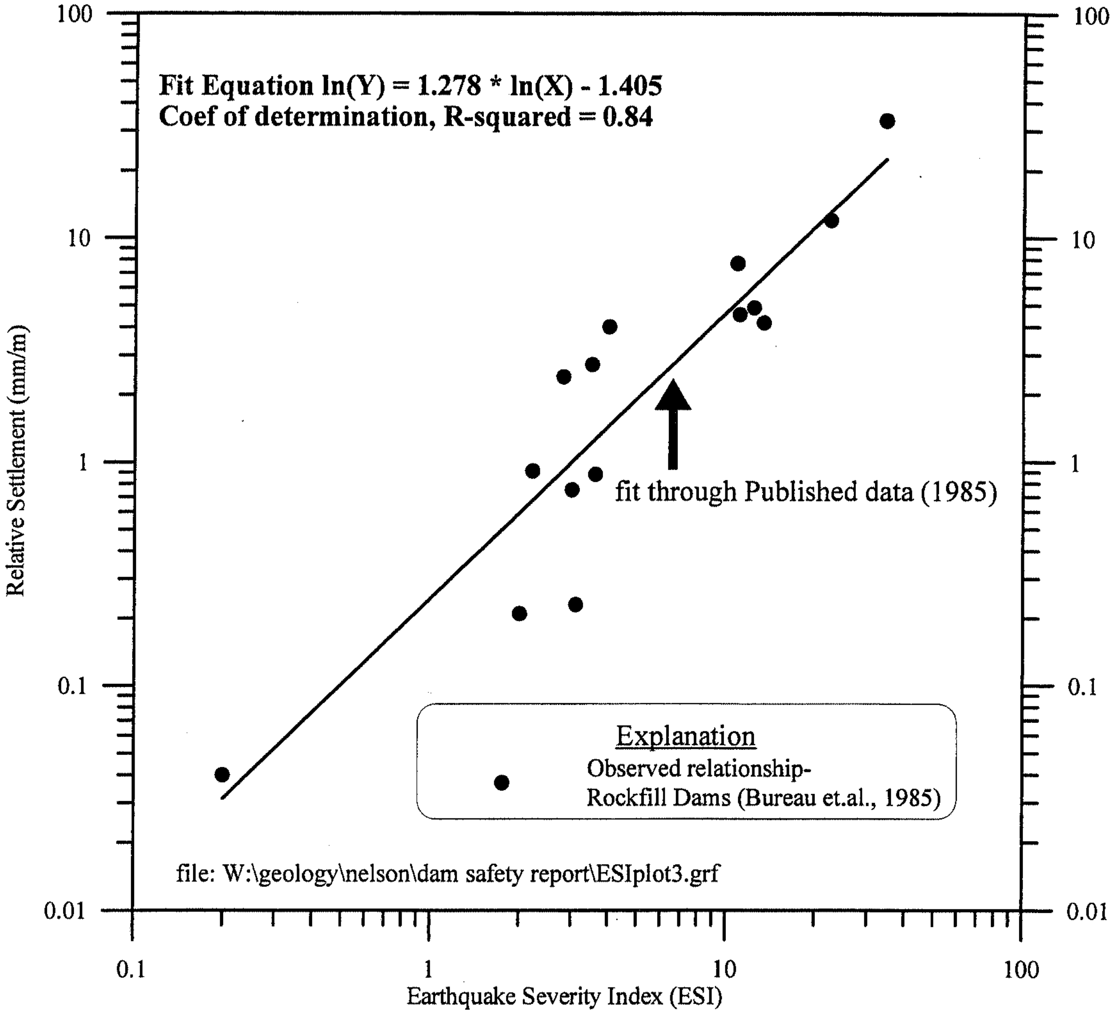


Figure C-5. Published data from Bureau and others (1985) relating Relative Settlement to Earthquake Severity Index

The line in Figure C-5 is a best fit line through the data based on a log function defined by the equation ln(Y) = 1.278 * ln(X) - 1.405 (where Y = Relative Settlement, X = ESI, and ln = natural log) with a coefficient of determination, R-squared = 0.84.

Figure C-6 shows a similar plot relating Relative Settlement and ESI for District dams and Austrian Dam. The figure shows a combination of actual recorded deformation and theoretically computed analysis based on dynamic finite element analysis. Three sets of data are indicated by different symbols including observed settlements due to the Morgan Hill Earthquake (stars), the Loma Prieta Earthquake (crosses), and estimated settlements based on seismic analysis by W.A. Wahler & Associates (1976, 1981) for a magnitude 8.5 earthquake (circles), termed the Maximum Credible earthquake, occurring on the San Andreas fault.

The line in Figure C-6 is a best fit line through the data based on a log function defined by the equation ln(Y) = 1.192 * ln(X) - 1.276 (where Y = Relative Settlement, X = ESI, and ln = natural log) with a coefficient of determination, R-squared = 0.73.

A comparison of Figures C-5 and C-6 suggests that District Dams, which are predominantly earthfill dams, share a common relationship with rockfill dams with respect to Relative Settlement and ESI. This is evident by the similar best fit lines through the two data sets. It should be noted that the best fit lines through the published data and the local data indicate relatively good correlations with R-squared values of 0.84 and 0.73, respectively.

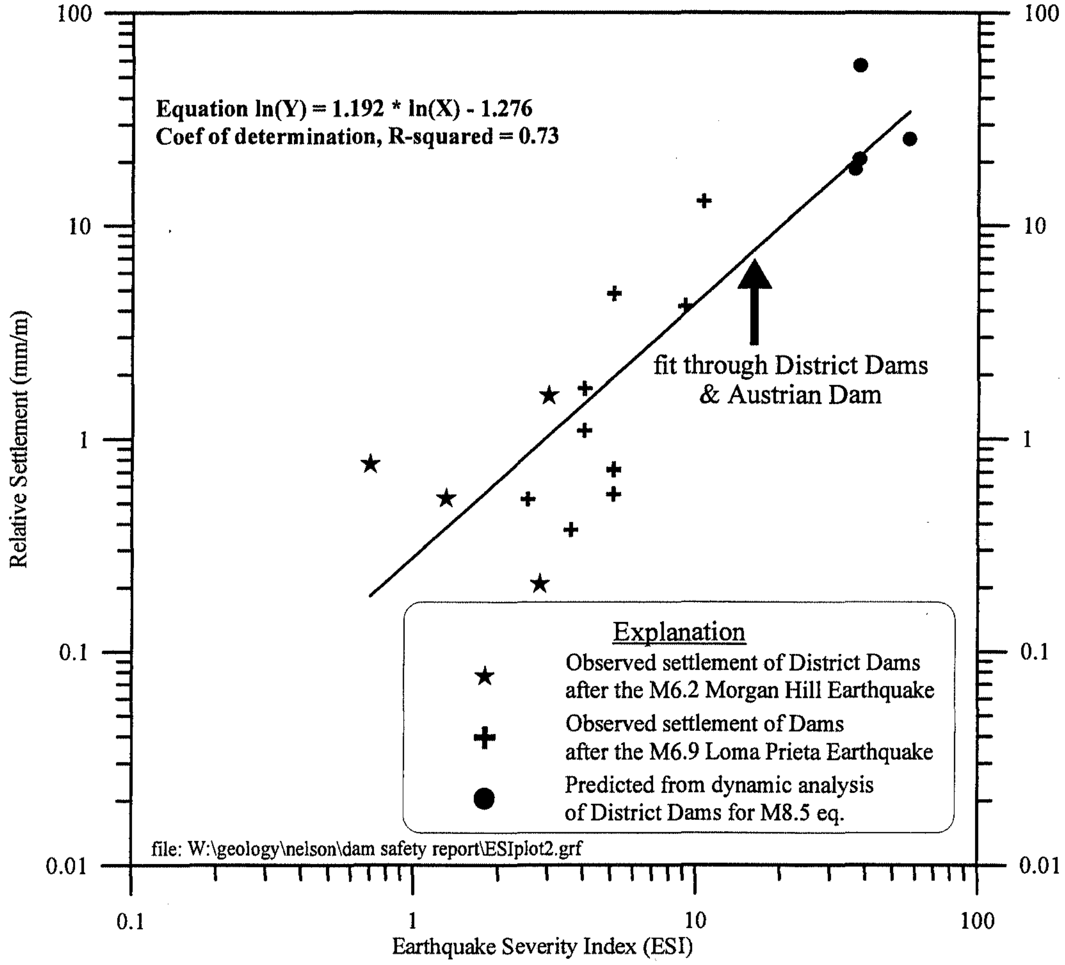


Figure C-6. Data from District Dams & Austrian Dam relating Relative Settlement to Earthquake Severity Index

Using the equation ln(Y) = 1.192 * ln(X) - 1.276 we calculated estimated earthquake-induced settlements due to likely future earthquakes. The results of these analyses are presented in the next section.

7.7.2 Results of Seismic Vulnerability Analysis

Table C-4 summarizes predicted settlements in inches for each earthquake scenario consisting of a M7.1 San Andreas Peninsula fault segment earthquake, a M6.9 Southern Hayward fault segment earthquake, a M7.0 entire Calaveras fault earthquake, and a M6.8 Monte Vista/Shannon fault earthquake.

Table C-4. Predicted Settlements from Various Earthquake Senarios

	Earthquake Senario			
	Peninsula San Andreas 7.1 earthquake	**Southern Hayward 6.9 earthquake**	**Entire Calaveras 7.0 earthquake**	**Monte Vista/Shannon 6.8 earthquake**
District Dams	estimated settlement inches	estimated settlement inches	estimated settlement inches	estimated settlement inches
Almaden	2.1	0.2	1.6	4.7
Anderson	2.1	0.5	14.8	3.1
Calero	1.6	0.2	2.0	4.3
Coyote	0.9	0.3	9.2	1.2
Chesbro	1.0	0.2	2.6	1.4
Guadalupe	3.8	0.3	1.7	7.7
Lenihan	13.0	0.5	1.6	8.7
Rinconada	1.7	0.1	0.4	2.5
Stevens Creek	7.8	0.5	1.1	8.2
Uvas	1.0	0.2	2.2	1.2
Vasona	1.2	0.1	0.3	1.9

Notes

1) acceleration values calculated using Attenuation relationship of Abrahamson & Silva (1997)
2) estimated Relative Settlement based on equation $\ln(Y) = 1.191900718 * \ln(X) - 1.276443754$ where Y = Relative Settlement and X = predicted ESI
3) Estimated settlement = Relative Settlment time the height of the dam
4) file: W:\geology\dam safety program\dam safety report\text\ESIpredict.xls

Table C-5. Dam Seismic Vulnerability Assessment to Future Earthquakes

Priority List of Dams Affected by a Future 7.1 San Andreas Peninsula Fault Segment Earthquake

District Dams	Predicted settlement S.A. Peninsula Earthquake (in)	San Andreas Peninsula Peak ground acceleration	Relative Vulnerability
Lenihan	13.0	0.70	A
Stevens Creek	7.8	0.64	A
Guadalupe	3.8	0.35	B
Almaden	2.1	0.26	B
Anderson	2.1	0.26	B
Rinconada	1.7	0.49	C
Calero	1.6	0.22	C

Priority List of Dams Affected by a Future 6.9 Southern Hayward Fault Earthquake

District Dams	Predicted settlement Entire Calaveras Earthquake (in)	Entire Calaveras Peak ground acceleration	Relative Vulnerability
Anderson	0.5	0.05	D
Lenihan	0.5	0.05	D
Stevens Creek	0.5	0.08	D

Priority List of Dams Affected by a Future 7.0 Entire Calaveras Fault Earthquake

District Dams	Predicted settlement Entire Calaveras Earthquake (in)	Entire Calaveras Peak ground acceleration	Relative Vulnerability
Anderson	14.8	0.74	A
Coyote	9.2	0.78	A
Chesbro	2.6	0.37	B
Uvas	2.2	0.30	B
Calero	2.0	0.30	B
Guadalupe	1.7	0.20	C
Lenihan	1.6	0.20	C
Almaden	1.6	0.23	C

Priority List of Dams Affected by a Future 6.8 Monte Vista/Shannon Fault Earthquake

District Dams	Predicted settlement MV/Shannon Earthquake (in)	MV/Shannon Peak ground acceleration	Relative Vulnerability
Lenihan	8.7	0.72	A
Stevens Creek	8.2	0.97	A
Guadalupe	7.7	0.92	A
Almaden	4.7	0.72	A
Calero	4.3	0.72	A
Anderson	3.1	0.25	B
Rinconada	2.5	0.97	B
Vasona	1.9	0.99	C
Chesbro	1.4	0.28	C

Notes:

Vulnerability Levels: A = high to very high; B = moderate to high; C = low to moderate; D = very low

file: W:\geology\dam safety program\dam safety report\text\damvulnerbility.xls

Table C-5 lists relative priority of the dams with respect to predicted disturbance, and thus vulnerability, for each earthquake scenario. It should be pointed out that the calculated settlement and acceleration values are for comparison purposes only, and are not intended to represent actual values from earthquakes. As a general guide we designate relative vulnerability levels A through D, with level A being high to very high vulnerability (greater than about 4 inches of settlement), level B moderate to high (between about 2 and 4 inches of settlement), level C low to moderate (between about 1 to 2 inches of settlement), and level D very low vulnerability (less than 1 inch of settlement).

The following is a list of District Dams vulnerable to the San Andreas Peninsula fault segment earthquake in decreasing order of disturbance: 1) level A vulnerable dams include Lenihan and Stevens Creek; 2) level B vulnerable dams include Guadalupe, Almaden, and Anderson Dams; and 3) level C vulnerable dams include Rinconada and Calero. For this scenario, predicted settlements range from 1.6 to 13 inches and peak ground acceleration range from 0.22g to 0.70 g.

District Dams do not appear to be significantly vulnerable to the Southern Hayward fault segment earthquake with only Anderson, Lenihan, and Stevens Creek Dams likely to experience minor level D settlements of about 0.5 inch and accelerations of 0.05 and 0.08g.

The following is a list of District Dams vulnerable to the entire Calaveras fault earthquake in decreasing order of disturbance: 1) level A vulnerable dams include Anderson and Coyote; 2) level B vulnerable dams include Chesbro, Uvas, and Calero; and 3) level C vulnerable dams include Guadalupe, Lenihan, and Almaden. For this scenario, predicted settlements range from 1.6 to about 15 inches and peak ground acceleration range from 0.23 to 0.78 g.

The following is a list of District Dams vulnerable to the Monte Vista/Shannon fault earthquake in decreasing order of disturbance: 1) level A vulnerable dams include Lenihan, Stevens Creek, Guadalupe, Almaden, and Calero; 2) level B vulnerable dams include Anderson and Rinconada; and 3) level C vulnerable dams include Vasona, and Chesbro. Predicted settlements range from 1.4 to about 9 inches and peak ground acceleration range of 0.28 and 0.99 g.

As seen from Table C-5 the most significant earthquake scenario that has the potential to impact most District Dams is by far the Monta Vista/Shannon fault 6.8 event. This earthquake is likely to produce the most dam disturbance due to particularly high accelerations and settlement.

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APPENDIX D

POST-EARTHQUAKE INSPECTION PROGRAM

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POST-EARTHQUAKE INSPECTION PROGRAM

1.0 Introduction

This appendix presents a discussion of various aspects of the District's post-earthquake inspection program. The reader is referred to Appendix C for basic earthquake information and section **5.0 Definitions and Terminology** of this appendix for explanations of pertinent earthquake and dam related terms.

The District's Emergency Operations Manual (SCVWD, 1976) covers earthquake emergency procedures. The general guideline stipulates that…"when a damaging earthquake occurs, those District facilities that present the greatest potential danger to the public will be inspected first, with those of less potential danger inspected later."

While the inspection of the dams, water treatment plants, and pump stations is automatic, the inspection of other facilities is undertaken on a priority basis. Priorities are established by the Emergency Preparedness Coordinator, in collaboration with senior management and the Chief Operating Officer, and based on consultation with District staff, coordination with the Santa Clara County Office of Emergency Services, and incoming information.

In 1983, just prior to the Morgan Hill earthquake of April 24, 1984, the District developed the present post-earthquake inspection program for critical facilities (Tepel and others 1984a). The Morgan Hill earthquake provided the first "real world" test of the inspection program. Post-earthquake inspections are an integral element in providing public safety in Santa Clara County. The key feature of this program is that a specific group of technicians are responsible for proceeding to assigned dams and inspecting them in accordance with written instructions and inspection forms tailored to each dam. The inspectors go to their assigned dams on their own initiative if one of the following two conditions occurs:

1) after any earthquake felt by the inspector or co-workers strongly enough to realize that it lasted about 10 seconds or more; or,
2) after an earthquake reported as magnitude 5 or greater and located in the County or within 20 miles of a District dam

Another part of the post-earthquake inspection program is the automatic dispatch of a helicopter to District headquarters by the District's contract flight service. This allows critical dams to be inspected within about two hours of the event.

A small earthquake (M_L = 4.9) occurred southwest of Gilroy on May 13, 2002. Since the earthquake was originally estimated at M=5.2, the post-earthquake dam inspections were automatically triggered. The closest District dam to the epicenter was Uvas Dam at 8.4 miles. Anderson Dam is located 13.9 miles from the epicenter. Due to the relatively close proximity of Anderson Dam to the epicenter, FERC requested that the District inspect the dam with a FERC inspector. It was decided to complete a Post-earthquake inspection report for this inspection. A copy of the inspection report, completed by Jim Nelson, Associate Engineering Geologist with the Dam Safety Section, Water Utility Engineering Unit, is presented at the end of this appendix. It represents a model for future post-earthquake inspection reports.

2.0 Inspection Considerations For All Dams

The following are critical issues to consider when performing post-earthquake inspections after significant seismic events (defined by the District as earthquakes with magnitude of 5 or greater and within about 20 miles of the facility).

2.1 Pre-Earthquake Inspection Activities

Each inspector should notify the EOC prior to leaving (or during travel to the site) for a post-earthquake dam inspection. This allows EOC officials to keep track of which dams are being inspected, insures that only one inspector is dispatched to each dam, and allows the EOC to further allocate inspectors if necessary. Once at the site, the inspectors should call EOC to state that they have arrived and are proceeding with the inspection. If possible, it is preferred that the inspector contact EOC by telephone because radio use allows second parties such as news reporters and general public potential access to the inspector's call-in. The concern is that second parties could be unnecessarily alarmed if they perceive that a dangerous situation exists. Consequently, if a phone is used, the tone and content of discussions relating to the safe condition of the dam should be as non-alarming as possible. Telephones are present in the control block houses at Almaden, Anderson, Coyote, Guadalupe, and Lenihan Dams. Inspectors should have keys to the block house on these dams.

2.2 Potential Access Problems

After a significant earthquake, inspectors should anticipate the likelihood that access roads to the dams have been damaged or made inaccessible by the presence of fallen trees, slope failures (landslides, rock falls, etc.), downed electrical lines, or bridge failures. Access to most District dams is provided by mountainous roads, which are notorious for having failures on the fill side (downhill side) of the road. After significantly wet winters it is not uncommon for half of the road to be gone in places due to slope failure. If the inspector is within a mile or so of the dam when their access is blocked, the inspector should hike to the facility. This would be quicker than bringing in a helicopter later.

2.3 Initial Inspection of Dam

Inspection of the dam should consist of an initial or primary inspection followed by a secondary more-detailed inspection. The purpose of the initial inspection is to look for immediate or impending failure, which should be reported immediately so that efforts can be made to minimize further damage to the dam, and to enable staff to prioritize efforts at multiple sites. Consequently, time is critical.

Even if there is no immediate danger, all inspectors should call the EOC after their initial inspection to give their preliminary findings. This will be followed by a more thorough secondary inspection, if warranted.

2.3.1 Look for Cracking and Landsliding

The primary inspection should consist of walking along the crest axis, zigzagging from downstream to upstream edges of the crest, looking for any cracking. Deformations of a dam will typically result in cracking, which could occur parallel (longitudinal) to the dam crest, perpendicular (transverse) to the crest, or at some angle. Arcuate cracking, or roughly following arcs as in a circle, typically indicate that major landsliding has occurred, or on the verge of occurrence.

2.3.2 Deformations Visible Along "Extended Line-of-Sight"

Evidence of possible deformation is made easier by observing physical features known to be relatively straight or linear over fairly long distances. These are referred to as "extended lines-of-sight." The inspector should become familiar with the degree of linearity of the established lines-of-sight at a particular facility prior to an earthquake so that they can differentiate between pre-earthquake and post-earthquake conditions. The following sections describe favorable extended lines of sight features at each dam.

The apparent line created by the intersection of the level dam crest with sloping surfaces of the upstream and downstream faces is a relatively good line-of-sight. This type of linearity is particularly good if it involves the upper edge of the concrete facing on the upstream side of the dam, like those present at Almaden Dam, Calero Main and Auxiliary Dams, Guadalupe Dam, and Stevens Creek Dam.

Another linear feature is created where the reservoir water level is in contact with the upstream concrete facing on each dam. Conversely, the contact line of the water surface is difficult to discern at Coyote and Anderson because they possess coarse rip rap on the upstream face.

The best place to evaluate the water-level line is to look at the concrete face from a vantage point slightly upstream of the dam crest. At some dams the water-level line appears curved relative to the construction joints on the concrete facing due to differential settlement and/or long-term lateral displacement (upstream/downstream direction) of the dam. The inspector should become familiar with the amount of "bend" in the water-level line prior to an earthquake so that they can differentiate between pre-earthquake and post-earthquake conditions.

2.3.3 Characteristic "Bends" of Water-Level Lines at Specific Dams

Almaden Dam: The water-level line has a subtle bend in the center of the dam relative to the left and right abutments.

Anderson Dam: Because coarse rip rap is present on the upstream face, a contact line of the water surface with rip rap is difficult to discern. Good lines-of-sight are the presence of guard-railing along the upstream and downstream edges of the dam crest, and the paved road along the crest.

Gaudalupe Dam: The water-level line has a more distinct bend in the center and right portion of the dam roughly in the form of an S-shaped curve. This movement most likely occurred in the upstream crest buttress following the 1989 Loma Prieta earthquake.

Calero Main Dam: The water-level line has a subtle bend in the center of the dam relative to the left and right abutments.

Chesbro Dam: The water-level line has a subtle upstream bulge in the center of the dam relative to the left and right abutments. A plot of monument survey data for Chesbro indicates that the center of the dam has moved upstream. Another favorable extended line-of-sight feature consists of a fairly long line of yellow metal poles present along the downstream face of the dam.

Coyote Dam: Because coarse rip rap is present on the upstream face, a contact line of the water surface with rip rap is difficult to discern. A good line-of-sight is the presence of the new line of survey monuments along the dam crest.

Lenihan Dam: Limited extended line-of-sight features. The straight centerline along the crest is about the only one. Others include the hydraulic conduit used for reservoir elevation measurements that runs from the block house down the upstream face of the dam and paved areas such the bike path (which provides a line-of-sight that is slightly curved) and Alma Bridge road along the crest. Paved areas include the road along the dam crest and the bike path that runs diagonally down the dam face.

Stevens Creek Dam: The water-level line has a subtle downstream bulge in the western half (left half) of the dam relative to the remainder of the dam. A plot of monument survey data indicates that the left half of the dam has moved upstream. Other favorable extended line-of-sight features consist of the following: the upstream and downstream edges the dam crest and the top of the concrete wall on the upstream face.

Uvas Dam: Because coarse rip rap is present on the upstream face, a contact line of the water surface with rip rap is difficult to discern. Good lines-of-sight consist of upstream and downstream edges the dam crest.

Vasona Dam: The contact between the concrete facing and the water surface has a subtle upstream curve.

2.4 Reservoir Affects - Standing Wave Oscillations or "Seiches"

A seiche (pronounced "saysh", Glossary of Geology, second edition) is a series of free or standing-wave oscillations (waves) created by the effects of seismic shaking on an enclosed or semi-enclosed basin. Inspectors should look for evidence of water level changes on the upstream face or along the shore of the reservoir that may indicate that a seiche has occurred. The presence of seiches will be more obvious on those dams with upstream concrete facing (Almaden, Calero Main and Auxiliary Dams, Guadalupe Dam, and Stevens Creek Dam) and much less obvious on those with rock faces. Seiches from a magnitude 7+ event on the Calaveras fault could produce waves up to 2 to 3 feet high at Coyote Dam. It is estimated that seiches from a magnitude 8+ event could produce waves up to 3 feet high. Minor seiches occurred in Anderson Reservoir during the Morgan Hill earthquake of 1984 and were actually recorded by the reservoir elevation recording devices. The maximum wave height was estimated at about 4-inches. Seiches from the 1989 Loma Prieta earthquake destroyed the three functional clarifiers at the District's Rinconada Water Treatment Plant.

2.5 Structures Sensitive to Strain

Rigid manmade structures such as concrete facing, asphalt paving, and guardrails are sensitive to strain induced by earthquakes. During a large earthquake, expect significant damage to the concrete facing on the upstream faces (Almaden, Calero Main and Auxiliary Dams, Guadalupe Dam, and Stevens Creek Dam), evident as overlapping panels, separated joints, and buckling. Even if severe damage has occurred to the concrete facing, it does not necessarily mean that the dam is in immediate danger. Asphalt paved areas are good for observing evidence of cracking.

2.6 Evidence of Cracking and Potential Landsliding

Because of their distinctive nature, cracking of the embankment usually looks worse than it actually is. Cracking is not a major concern as long as rainwater does not fill the cracks. Cracks usually disappear relatively quickly, especially as a result of significant rainfall. Soon after the Loma Prieta earthquake, rainfall erased the evidence of some of the cracking observed on District dams within about 36 hours of the event. Consequently, location of cracks should be documented as soon as possible. Observed cracks should be marked by pin flags (and photographed) so that their location can be mapped at a later time. It is particularly important to mark the extreme ends of cracks with flagging or spray paint to detect whether cracks are growing as a result of progressive landslide movement or due to new crack propagation from aftershocks.

A potentially more dangerous situation is initiation of landsliding on either the upstream or downstream faces of the dam, which will be evident as arcuate (arched or curve shaped) cracks at the outer edges, connected by a longitudinal crack (oriented parallel to the crest). The combination of two outer arcuate cracks connected by a longitudinal crack probably marks the head of a landslide.

There are two categories of cracks depending on their orientation relative to the dam crest. Transverse cracks are those that are perpendicular or at an approximate right-angle to the crest. This type of cracking is typically due to tension or separation. Longitudinal cracks are roughly parallel to the crest and are produced when there is a lack of lateral support resulting in a landslide. Transverse (tension) cracks will generally be open, with no vertical displacement (up-down component of movement), while longitudinal cracks may exhibit predominantly vertical displacement.

Another factor to consider is the reservoir level. During times when the reservoir level is high, and the upstream shell is saturated, the dam is more likely to experience significantly more cracking and possibly earthquake-induced movement. For earthquakes with a magnitude greater than M=6.5 occurring within 20 miles of the structure, there is a possibility of cracking. As a result of the 1989 Loma Prieta, several dams experienced cracking up to 3 to 4 inches wide. Landslides are more likely to occur on the upstream face below water level and such movement may be difficult to observe. If the headscarp is above the reservoir elevation, it will appear as a sharply steepened surface. Fortunately, during the Loma Prieta earthquake most District reservoirs were only about 1/4 to ½ full due to the preceding drought. At Guadalupe, a landslide failure began to occur within the upstream buttress fill during the Loma Prieta earthquake, but then stopped propagating when the earthquake shaking stopped. The landslide cracking, although relatively deep was subsequently repaired by excavation and replacement.

2.7 Floating Hydraulic Oil

The presence of hydraulic oil floating on the reservoir surface could indicate either leakage from hydraulic lines to the control valves or slope failure below the water line has ruptured the lines. Significant leakage may mean that the control valves will not operate, which is critical if the reservoir needs to be lowered in response to an emergency condition.

2.8 Areas with Historic Seepage

Seepage patterns may change or seepage may increase as a result of strong earthquake shaking. Cracks in bedrock can open up, resulting in increased seepage. Apparent increase in seepage conditions was detected at Almaden Dam as a result of the Loma Prieta earthquake. This seepage has been attributed to opening of bedrock fractures.

The inspector should become familiar with relative discharge rates from weirs and other established seepage points, which will likely vary depending on the reservoir level. More seepage typically occurs at higher reservoir levels due to the higher head. The discharging seepage should be inspected for the presence of soil particles as evident by clarity of the water. Cloudy water indicates the seepage is sufficiently high that it may be removing soil particles (clay, silt, sand) from the dam. This condition (known as piping) is a potentially very dangerous and should be immediately reported. Several dams have failed due to piping.

The inspector should note any changes in seepage flow clarity with time. At dams with significant historic seepage, the inspector should use a spray paint can to mark the highest level on the dam face (find a suitable boulder to paint) where water is visibly discharging and then monitor any changes with time. Look to see whether water in the weir box has become cloudy, which could indicate internal seepage and potential danger to the safety of the dam. Collect a sample of water directly from the pipe discharging water to the weir in a container or hard hat to look for soil particles (particularly sand) that could indicate evidence of internal erosion or piping. It is possible to have apparently clear water in the weir box, but detect sand particles coming from the discharge pipe. This could indicate possible adverse conditions for dam safety. The following is a brief description of normal seepage conditions for each District Dam.

Almaden Dam Seepage: Localized seepage is present at the left abutment groin area near the downstream toe of the dam. A seepage weir is situated adjacent to (directly west of) the outlet control block house. This weir does not collect all of the seepage as some water can be seen flowing around the eastern side of the block house. The inspector should use a spray paint to mark the highest level on the dam face (find a suitable boulder to paint) where water is visibly discharging.

Anderson Dam Seepage: Seepage from the dam has never been observed. Seepage does occur, however, beneath the spillway along the bedrock foundation in response to changes in reservoir elevation. Recently, two new seepage weirs were constructed at the spillway terminus. The flow rate monitored from these two weirs varies dramatically with increasing reservoir level.

Calero Main and Auxiliary Dams and Fellows Dike: Localized seepage is present at the right abutment groin area and on the downstream of the main dam. The seepage appears to increase when the reservoir level is above about 470 feet. Two seepage non-functional weirs situated directly downslope of the right abutment groin area have been abandoned. The Auxiliary Dam and Fellows Ranch Dike have not had significant seepage. Neither facility has seepage weirs.

Although the Auxiliary Dam appears to be a very small dam, it is relatively significant because it retains as much as 1/2 to 3/4 of the full reservoir capacity.

Fellows Ranch Dike is situated at the southwestern end of the reservoir. The District does not have any records on how the dike was constructed. A recent investigation indicated the dike was not constructed to conform to suitable geotechnical standards. Potentially liquefiable soils were identified along the portion closest to Calero Creek and the District is currently (February 2003) developing plans to excavate and replace the questionable materials and bring Fellows Dike up to current safety standards.

Chesbro Dam Seepage: Has not experienced significant historic seepage. Some minor areas with green vegetation indicative of seepage have been evident along the downstream face near the left abutment when the reservoir is high. A discharge pipe is present in the bottom of the spillway that discharges water from the foundation of the dam during the rainy season.

Coyote Dam Seepage: Although a discharge weir is present at the toe of the dam that discharges water from the foundation and the dam Coyote Dam has not experienced significant historic seepage. The two spillway walls are founded in bedrock. Because the walls are located in bedrock, new seepage emanating from the spillway walls or slabs is probably not related to dam integrity and therefore is not likely serious.

Guadalupe Dam Seepage: Has not had significant seepage. Minor seepage has been noted at the right groin, about 50 feet downslide of the crest and minor seepage near the downstream toe. During a July 26, 2001 site visit, we observed water-loving plants in the vicinity of the right groin area, but we did not obvious free water. We also observed minor seepage from an area directly north of the outlet valve block house at the downstream toe of the dam.

Lenihan Dam Seepage: Has not had significant seepage. Localized seepage is present during the winter on the downstream face due to the presence of shallow permeable layers formed during construction. During the rainy season, weeping areas can be seen covering the bike path. Seepage patterns may change or seepage may increase as a result of earthquake shaking. Cracks in bedrock can open up, resulting in increased seepage.

Rinconada Treated Water Reservoir and Dam Seepage: Seepage at this facility has been very small and may have been due (in part) to leaking water pipes.

Stevens Creek Dam Seepage: Has not had significant historic seepage. Some minor areas with green vegetation indicative of seepage have been evident along the downstream face near the right abutment when the reservoir is high. A discharge weir is present at the toe of the dam that discharges water from the foundation and the dam.

Uvas Dam Seepage: The dam has had a long history of seepage and attempted grouting to limit the seepage. The last grouting program (1996) focused in the right abutment and appears to have been successful in stopping the seepage in the area. The left (east) abutment and foundation are still leaking with weirs 1 and 3 measuring maximum flows of about 65 and 35 gpm at full reservoir level.

Vasona Dam Seepage: There is no major seepage visible through the dam.

2.9 Rockfalls

Rockfalls are likely to occur at all District dams because of the presence of near vertical rock slopes adjacent to the dam and/or spillway. These slopes were produced during construction of the dams and appurtenant structures. This type of slope failure would probably not be a critical factor in the safety of the dam, but would be worth reporting.

Almaden Dam: Rock falls of the rock cliffs immediately upslope of the right (eastern) walls of the spillway are likely.

Calero Main and Auxiliary Dams and Fellows Dike: The inspector can expect to observe rock-falls from the rock cliffs on the prominent knoll immediately upslope of the Main Dam right abutment. This knoll separates the Main Dam from the spillway. Some minor landsliding could occur upslope of the Almaden-Calero Canal at the left abutment of the Main Dam. This will not likely be critical to dam safety.

Coyote Dam: The two spillway walls are founded in bedrock and it is known that the Right Abutment fault passes diagonally across the spillway. Although this fault is not considered to be active, the potential exists that it could experience sympathetic movement during a large earthquake on the active trace of the adjacent Calaveras fault. There is also a high probability of rock falls from the rather steep slope behind the spillway.

Lenihan Dam: It is highly likely that rock falls will occur from the rock cliffs immediately upslope of the right abutment and upslope of the left spillway wall and from the rock cliff below the end of the spillway.

Stevens Creek Dam: It is highly likely that rock falls will occur from the rock cliffs immediately upslope of the right spillway wall.

Uvas Dam: It is highly likely that rock falls will occur from the rock cliffs immediately upslope of the left abutment across from Uvas Road. It is not clear that such rock falls would impact the dam.

Vasona Dam: There are no rock cliffs adjacent to Vasona Dam.

2.10 Spillway Damage/Seepage

In the spillway, look for pop-outs of the concrete panels on the floor and wall of the spillway. Pop-outs could inhibit the flow of water through the spillway and could lead to spillway damage by erosion. The level of effort may not warrant going into the spillway. The inspector may be able to view the spillway from above. Damage to the spillway is more critical during the winter than the summer due to the potential for spilling of the reservoir after significant rainfall events.

The inspector should become familiar with seepage patterns in the spillway at each dam. Generally speaking, the potential for serious seepage conditions depends on which wall is seeping water. Typically, the wall furthest from (opposite) the dam is founded in bedrock, while the nearest wall is supported by the dam. Discharging water from the opposite bedrock wall is probably not serious. Discharging water from the wall supported by the dam, however, could indicate a more serious condition with respect to dam safety. Some seepage may occur from spillway walls, either historically or as a result of an earthquake. Areas that have had long-term ongoing seepage are evident by growth of algae on the concrete wall and/or floor.

Anderson Dam Spillway – Two new "V" notch weirs have been installed to collect the spillway underdrain seepage that occurs at Anderson Dam. The weirs have not yet (Winter, 2001) been subjected to full reservoir conditions.

Almaden Dam Spillway - Historic seepage has occurred from the western spillway wall above the 5th floor panel downstream of the spillway weir. Recently, offset of about 2" to ½" has been observed at the 2nd and 7th right panel joints, respectively.

Guadalupe Dam Spillway: No historic seepage

Calero Main and Auxiliary Dams and Fellows Dike: The two walls of the spillway at Calero Main Dam are founded in native material and/or bedrock. The spillway is unique because it is situated several hundred yards from the dam. Minor damage of spillway panel has occurred on the 1) 4th through 7th panels from the spillway toe along the right wall, 2) 1" to 2" offset that begins 2 panels downstream from a previous repair to the right spillway wall above the stilling basin, and 3) the 4th through 7th panels from the spillway toe are protruding ½" from the wall.

Chesbro Dam Spillway: The two walls of the spillway at Chesbro Dam are founded in native material and/or bedrock.

Stevens Creek Dam Spillway: The spillway walls are founded in embankment material (left spillway wall) and bedrock (right spillway wall). Evidence of new seepage after an earthquake from the left wall could relate to potential damage of the embankment and has the potential to be related to integrity and safety of the dam. Because the right wall is located in bedrock, new seepage here is probably not related to dam integrity and therefore is not likely serious.

Uvas Dam Spillway: Uvas spillway has not exhibited historic seepage. However, the rock slope southwest of the spillway (between the spillway and embankment) exhibits some seepage, which normally occurs when the reservoir is high.

Vasona Spillway: Other than minor gate leakage, there has not been a history of spillway seepage at this facility.

2.11 Calero-Almaden Canal

After a significant earthquake in the vicinity of Almaden and Calero Dams the potential exists that the Calero-Almaden Canal would be used to drawdown water from Almaden reservoir, especially if the safety of Almaden Dam is in question. In such a case the continued operation of Calero-Almaden Canal is critical. Consequently, blockage of the canal due to earthquake induced landsliding while water is being discharged disrupts operations and has the potential to flood residential parcels situated on the downslope side of the canal. During canal operation someone from O&M is typically assigned to inspect on a regular basis.

2.12 Other Miscellaneous Impacts

Almaden Dam: The San Jose Water Company has water storage tanks and associated pipelines downstream near the left abutment of the dam. It is possible that these tanks and pipelines could fail and leak during significant seismic events. This would not likely impact the safe operation of the dam.

Lenihan Dam: The San Jose Water Company has two 30-inch diameter water lines running parallel to the dam crest along the immediate upslope edge. Substantial movement of the crest could cause failure of the water lines resulting in significant erosion of the dam. Consequently, it is critical that for the inspector to look for evidence of leakage along the water lines.

Based on experience from the Loma Prieta earthquake, the inclinometer casing in LDI-1 along the crest of the dam, and/or LDI-2 on the downstream face, may be sticking out of the ground due to seismically induced settlement of the dam. It is estimated that Lenihan Dam could settle 2 to 3 feet during a significant earthquake (8+ magnitude). Similar displacements could occur at other dams.

Look for evidence of cracking on the relatively new shotcrete facing on the east and west sides of the spillway entrance. Several fill slopes were added to the upstream face of the dam to provide access for recent drilling operations. Look for evidence of cracking of fill due to accelerated compaction.

It is hypothetically possible that the Alma Street Bridge, located upstream of the spillway, could fail during a significant earthquake. Such a bridge failure, unless the reservoir is full and there is a chance for a spill, may not seriously affect the safe performance of the dam. However, failure of the bridge will affect access to the dam. During a large seismic event, the entire spillway could fail. Even if the spillway were to fail, we expect the dam to remain intact. It should be noted that the left abutment of Lenihan begins on the east side of the bridge, not the west side of the bridge as one might expect. The left abutment is founded in a prominent bedrock ridge that separates the spillway and the embankment.

Finally, the inspector should walk down to the outlet pipe to observe seepage exiting the pvc pipe weepholes in the gunnited outlet channel lining. Looking downstream at the outlet pipe, the pvc pipe is at the "7-o'clock" position. Observe for evidence of cloudy/muddy water indicative of potential piping failure.

Uvas Dam

In the past, staff has used the "RPs" (series of vertical metal pipes with elevation markings) along the upstream right groin to estimate water levels. Staff said that the RPs need to be replaced where missing and re-surveyed.

For the initial inspection, look at the lower downstream berm for cracking and seepage.

Look for any water flowing from the transfer pipeline structures that pass from the outlet structure through the downstream (southeastern) end of the spillway channel.

Look around and in the block house of the outlet structure to look for evidence of seepage that might be occurring along the outlet pipe.

Walk along the outlet pipe concrete encasement to look for evidence of leakage and or seepage. It has been noted that during parts of the year clear water can be seen flowing over the concrete encasement.

Go to weirs 1, 2, and 3, and the toe drain weir. Take a reading of the staff at each weir. Look to see whether water in the weir box has become cloudy, which could indicate internal seepage and potential danger to the safety of the dam. Collect a sample of water directly from the pipe discharging water to the weir in a container or hard hat to look for soil particles (particularly sand) that could indicate evidence of internal erosion or piping. It is possible to have apparently clear water in the weir box, but detect sand particles coming from the discharge pipe. This could indicate possible adverse conditions for dam safety.

3.0 Pumping Stations And Water Treatment Systems

The District owns and operates various pumping stations and sectionalizing valves to control water transport from several sources to the various reservoirs, treatment plants, and treated water retailer. The following paragraphs describe the general water transport system hydraulics and associated critical pumping stations, pipelines, and valves.

The basic mechanism driving the flow of water from the USBR San Felipe Division Project (Pacheco-San Felipe-Santa Clara Conduit) is the difference in elevation between the high point of the system, which is the regulating tank above the Pacheco Pumping Plant at San Luis Reservoir, and the elevations of points downstream where the water is no longer under pressure and finds a free surface with the atmosphere. Gravity feed occurs because of the water's free surface at its high point in the Pacheco tank being significantly above the water surface elevation at Anderson Reservoir at about the half-full elevation, so gravity feed is actually from the Pacheco regulating tank, and the water passes Coyote Pumping Plant and other low points on the way. The pumping plant is used to increase the head in supplying water to the Almaden Valley Pipeline (AVP), which can then be delivered to the District's two largest treatment plants - Santa Teresa and Rinconada, and other distribution facilities.

The Santa Clara Conduit Calaveras fault crossing is situated directly south of San Felipe Lake and Pacheco Pass and consists of inlet and outlet structures (vaults) that house the Santa Clara Conduit pipeline and valves. As the 96-inch diameter pipeline from Pacheco Pass runs through the inlet structure, the pipe branches into two parallel 66-inch conduits. The two conduits run for about 700 meters and cross the actively creeping trace of the Calaveras fault. This design feature allows for possible future repair of the pipeline if cumulative fault movement damages the pipe. Water flow can continue through one conduit while the other conduit is being repaired, and vice versa. At the outlet structure west of the fault, the two parallel 66-inch conduits again merge into a 96-inch pipeline.

To accommodate fault movement, articulated joints were constructed at each pipe joint, which is every 10 feet. Internal "Weco seals" (comparable to dresser couplings that work like hose clamps) are used at several joints to seal the pipe joints. No leakage has been evident along the pipeline since the Loma Prieta earthquake. Survey monuments have been placed along the top of the two pipes for periodic monitoring of relative displacement, although they are not currently effective.

Formerly, water transport from the Santa Clara Conduit passed through the Cross Valley Pipeline (CVPL) which ended at and emptied directly into Calero Reservoir. Calero in turn was the sole supply to the AVP. The Calero Pipeline was built around 1990 which bypasses Calero Reservoir, so now the AVP is fed directly from the Calero Pipeline from the CVPL. Calero Reservoir now rarely receives much flow from the CVPL. The Coyote Pumping Plant (like most raw water controls, it is operated remotely) is controlled by the Raw Water System Operators through a computerized Supervisory Control And Data Acquisition (SCADA) system at the Rinconada Water Treatment Plant.)

Sectionizing Valves SV-1 and SV-2 are used to control water between the Pacheco Pumping Station and the Cross Valley Pipeline. Flow to the North through this region is by gravity. Specifically, the valves are used to 1) isolate the flow from the pipeline to specific areas; 2) fill Anderson Reservoir by closing the valves; and 3) to fill the pipeline slowly.

Vasona Pumping Plant is used to control water diversion activities between the Central Pipeline and the Almaden Valley Pipeline, which are in turn associated with various water sources and treatment plants. The valves are used to configure and divert the flow from the two pipe lines. The pumping and valve operations at the facility are controlled by the Raw Water Control System at the Rinconada Water Treatment Plant.

3.1 Response of Pumping Stations and Sectionalizing Valves to Earthquakes

Critical structures at these facilities include pumps and piping. The pumps are sensitive to alignment of internal components and to alignment with piping. Consequently, after significant seismic shaking, an inspection should be made to determine whether the pumps are still standing and are able to function properly. At most of District facilities, the pumps appear to be adequately bolted to the concrete base.

Another major concern is differential movement between building/vault walls and entrance holes for the pipes during seismic loading. As the various seismic waves pass through the inlet and outlet structures from all directions, an unsupported pipeline will tend to flex upward and downward as it responds to compression. Favorable construction practices evident at most facilities include 1) stabilization of vault structures by a saddle support structure; 2) reinforcement of pipes with pre-stressed wire and concrete covering; and 3) encapsulation of pipes in massive concrete making it less likely for breaks to occur in the bond between the pipe and the concrete. Consequently, the pipelines and concrete housing structures are more likely to move as a single unit. Because structures housing the pumping stations are fairly low, one would not expect there to be resonance effects between soil and heavy equipment. Resonance usually involves the swaying of tall buildings when their resonance matches those of the underlying thick soil.

Initial Post-Earthquake Inspection of Pump Stations and Sectionizing Valves

The initial inspection should consist of walking around the main building that houses the pumps, and inspecting critical features inside the building. Critical features include the pumps and piping. Look at the pumps to see whether they are still standing and are able to function properly. Another consideration is to observe whether overhead objects have fallen during earthquake shaking.

An inspection should be made of the valve yard to look for evidence of significant leakage from the piping and valves.

Post-earthquake inspectors should become familiar with normal cracking patterns in the plant building as they presently exist, so that a comparison can be made with any possible new cracks formed as a result of seismic shaking.

Failure of Pacheco Regulating Water Tank

During seismic loading, typical water tank failures include breaks where a pipe enters the side of a tank. This is probably not critical if the pipe enters the top of the tank. Also, significant bulging can occur at the base of the tank known as an "elephant foot." Because the tank is situated directly upslope of Pacheco Pumping Station, it will be easy to see and detect any leakage of water.

3.2 Pacheco Pumping Station

The primary inspection should consist of looking at the water tank upslope of the facility, walking around the main building that houses the pumps, and inspecting critical features inside the building. Critical features north of the main building include a 500-gallon propane tank and associated gas piping for the emergency power generator.

Because the entire facility appears to be on bedrock and fairly stable rock slopes are present upslope of the tank, one would not expect significant damage to cuts, the tank, or associated buildings.

Inspections in the main building should consist of looking at the pumps to see whether they are still standing and are able to function properly. Look in the ASD room to see if metal cabinets for the motor control electrical circuits are still standing. These cabinets are not securely fastened for seismic shaking hazards. The suspended floor has not been seismically retrofitted.

Inspect the condition of the 500-gallon propane tank and associated gas piping to the emergency generator to determine whether the tank or pipe is leaking. Localized, minor corrosion is present along the gas piping and on the tank footings. There are current concerns regarding the ability of the tank footings to withstand seismic shaking hazards.

Pacheco Sectionalizing Valve

Another secondary feature to inspect would be the Pacheco Sectional Outlet facility, situated west of the pumping plant along Pacheco Pass (Highway 156). This facility has a butterfly valve to open or close the pipeline and a diversion pipe around the valve to control water flow.

3.3 Coyote Pumping Plant

This facility consists of a pumping plant that contains six 2000 horsepower pumps and associated valves to direct water to various places. The power substation at the plant is only used to power the facilities.

The building housing the pumps at Coyote appears to have been designed with a large factor of safety regarding seismic stability. The plant building actually consists of two structures aligned end to end. The Bureau of Reclamation has placed brass monuments inside the building on the floor slabs of each structure where they are joined. It appears likely that the building was constructed as two separate structures so that the long axis of the building would not be adversely affected by typical wavelengths of Rayleigh seismic (surface) waves.

During the Loma Prieta earthquake, the facility probably experienced significant accelerations and appears to have responded favorably. No known damage occurred during the Loma Prieta earthquake. Although Coyote Pumping Station has not been subjected to significant earthquakes on the Calaveras and Hayward faults, one would not expect wholesale failure of the facility.

Because the entire facility appears to be on fairly stable sediments and no sloping ground are present upslope of the facility, one would not expect significant damage to associated buildings during seismic loading. Excavation of the foundation during construction of the pump building appears to have resulted in placement of the building on firm ground.

Following a major earthquake, inspections similar to those described in Section 3.2 for the Pachecho Pumping Station should also be completed at the Coyote Pumping Plant.

3.4 Vasona Pumping Plant

This facility consists of pumping plant and associated valve yard, pilot plant, and a meter shop. The pumping plant and associated valve yard are the focus of the inspection orientation. This is a critical facility because it is used to control water diversion activities between the Central Pipeline and the Almaden Valley Pipeline, which are in turn associated with various water sources and treatment plants. The valves are used to configure and divert the flow from the two pipe lines. The pumping and valve operations at the facility are controlled by the Rinconada Water Treatment Plant.

Historically, there has been normal seepage of groundwater into the walls of the basement floor that houses the large pipes. Also, the walls of the various valve vaults seep water during the winter. Some of the vaults have sump pumps. Grouting has been placed to seal the gap between the pipe and the walls of the valve. Occasionally, the grout is "blown out" by the seepage.

During the Loma Prieta, the facility probably experienced significant accelerations and appears to have responded favorably. The only reported damage was to a new lathe that had not yet been bolted to the floor and cracking of two tanks that hold polymer. The tanks had cracked on the lower 1/3, at the sides of each tank. New tanks have replaced the damaged ones. The new tanks have seismic restraints and double containment.

Because the entire facility appears to be on fairly stable sediments and no sloping ground are present upslope of the facility, one would not expect significant damage to associated buildings. Excavation of the foundation during construction of the two floors of the pump building appears to have resulted in placement of the building on firm ground. There does not appear to be a need for complete redesign of the facility, just recognition of the potential hazards during seismic shaking.

Following a major earthquake, inspections similar to those described in Section 3.2 for the Pachecho Pumping Station should also be completed at the Vasona Pumping Plant.

3.5 Santa Clara Conduit Calaveras Fault Crossing

The area surrounding the facility consists of very flat lake sediments that are underlain by soft compressible soils. Periodic flooding of the area during very wet winters results in the rise of water over the tops of the inlet and outlet structures.

The primary inspection should consist of looking down into the inlet and outlet structures (vaults) or entering the structures if necessary. Generally speaking, the vaults are considered a confined space and entering them would require confined-space training.

Critical features in the vault structures include valves and pipelines. During the initial inspection, look for evidence of significant leakage from the pipes and/or valves. Look at unsupported equipment to see whether it still standing and is able to function properly. A major concern is differential movement between vault structure walls where the pipeline enters and exits the vault structure. Where the two 66-inch conduits exist and enter the inlet and outlet structures, the pipe and flange have been embedded in the concrete of the structures. This design is favorable with respect to earthquake shaking because the pipe and flange become part of the structure.

Although the entire facility is located on fairly soft compressible soils, one would not expect significant damage to the facility. There does not appear to be a need for complete redesign of the facility, just recognition of the potential hazards during seismic shaking.

During our inspection we observed one strong motion instrument in the inlet structure and one strong motion instrument in the outlet structure. These instruments were originally maintained by the Bureau of Reclamation but, due to lack of funding and maintenance personnel are now inoperable.

As discussed below, it is highly probable that a major earthquake emanating on the Calaveras fault in reasonably close proximity to the CFI/CFO crossing may produce failure of one or more of the pipe joints. Any post-earthquake inspection needs to look for field evidence of joint failure.

Response of Facility to Loma Prieta and Morgan Hill Earthquakes

During the Loma Prieta earthquake, the facility may have experienced slight seismic accelerations and appears to have responded favorably. As the various seismic waves pass through the inlet and outlet structures from all directions, an unsupported pipeline will tend to flex upward and downward as it responds to compression. The pipelines in the vault structures are stabilized by a saddle support structure. After the Loma Prieta earthquake, the most significant problem was differential settlement of the inlet structure and leakage of water from one of the pipes on the bifurcation side of the inlet (CFI) structure. The leak was repaired using a Weko seal.

3.6 Sectionalizing Valves SV-1 and SV-2

These facilities consist of vaults that house the Santa Clara Conduit pipeline and valves for controlling flow through the pipeline. SV-2 is located at the eastern end of San Martin Avenue near the City of San Martin and SV-1 is located off of Leavsely Road, east of Gilroy.

The primary inspection should consist of looking down into the vault or entering the vault if necessary. Generally speaking, the vaults are considered a confined space. During our inspection, an elevator was present for gaining access to the vault.

Critical features in the vault include valves and pipeline. During the initial inspection, look for evidence of significant leakage of the piping and/or valves. Look at unsupported equipment to see whether it still standing and is able to function properly. A major concern is differential movement between vault walls where the pipeline enters and exits the vault. It is not expected that resonance would develop between soil and relatively heavy mass of the pipeline and valving. Resonance usually involves the swaying of tall buildings when their resonance matches those of the underlying thick soil.

Another consideration is the falling of overhead objects during earthquake shaking.

During the Loma Prieta, the facility may have experienced moderate seismic accelerations and appears to have responded favorably. As the various seismic waves pass through the structure from all directions, an unsupported pipeline will tend to flex upward and downward as it responds to compression. The pipeline in the vault is stabilized by a saddle support structure. During the Loma Prieta earthquake, the most significant problem was with the pressure relief valves staying open.

Because the entire facility appears to be on fairly stable sediments and no sloping ground are present upslope of the facility, one would not expect significant damage to associated buildings. Excavation of the foundation during construction of the vault appears to have resulted in placement of the vault into firm ground.

4.0 Post-Earthquake Inspection Procedure Kits

The following discussion pertains to post-earthquake inspection kits which should be developed for each dam. It would appear that the kits should be stored at the site so they are readily available to whoever shows up to complete the inspection. Some identification cards and placards, however, will be necessary if inspectors have to take their own vehicles for inspections. The inspector should have photographs that show the condition of pertinent features before significant earthquakes, so the pictures can be compared to post-earthquake conditions.

A suggestion has been made that the District should put permanent numbering on each concrete panel of the spillway and the upstream dam face so that inspectors could easily describe the location of damaged panels to others. This work has been scheduled for completion.

It would be advantageous to stockpile the differing tools / inspection packages at either the up stream valve vault or in the "crest telemetry block house". This same storage area would have a log book and those responsible staff people could inspect this material once a month during their normal duties. These same staff people would have to have a key to the block houses as well. Also, should the quake happen at night or very early in the morning hours, it would appropriate to stock flares at the site. Should a helicopter have to land on the dam (with senior engineers and geologists), it would certainly assist the pilot in having a high lighted area,

4.1 Dam Inspection Kit Needs - Suggested

1. Include photographs of general area for comparative purposes.

2. Vicinity maps showing with main and secondary access roads highlighted for each dam.

3. Print forms on stock that can be written on when wet.

4. Include a kit contents checklist.

5. Diagram showing dam and instrumentation locations with terms left, right, piping, traverse, longitudinal, etc.

6. Include pens, pencils, disposable flashlight and disposable camera.

7. Print instruction materials on water-resistant stock.

8. Pin flags and spray paint cans for documenting and locating suspicious seepage and cracks.

9. Include maps showing areas with historic seepage, so inspector has something to compare to actual field conditions.

10. Provide photographs showing examples of seepage at pertinent times of the year

11. Provide a list of terms and their definition relating to the dam and earthquakes.

12. Provide historic data plots of gauge reading of weirs (in addition to discharge readings) so that they know what is "normal" during various times of the season.

13. Provide a list of phone numbers to be called to report post-earthquake observations. Inspectors need to know the hierarchy of which responsible individuals to notify during earthquake emergencies.

4.2 Other Post-earthquake Related Issues

1. Place permanent numbering on upstream concrete face panels to identifying damage or seepage.

2. The District may consider periodically performing surprise earthquake response drills so that inspectors will be comfortable with performing post-earthquake inspections.

4. Contact the Bureau of Reclamation to find out what design earthquake was used for the facility and compare to present state of the practice.

5. Find out specifics of the seismometers present at Bureau of Reclamation sites (Pacheco Pump Station, Calaveras fault crossing, Coyote Pump Station).

6. Find out from the Bureau whether they have strong motion records from the instruments at various facilities from the Loma Prieta earthquake. The following name and phone number were on the strong motion instruments at the Calaveras fault crossing: Dave Copeland, (303) 236-4196.

7. Maps of facilities such as the pumping plants could be taken from HMMP (hazardous materials management plans) available from Rinconada.

4.3 Dam Inspection Kit Needs – Status as of February 2003

It is recommended that the Dam Inspection Kits be upgraded and finished in FY04 for availability to all post-earthquake inspectors. The following items still need to be prepared and stored in a suitable weather proof container at each of the dams.

1. Guidelines for Inspection of Dams Following Earthquakes by United States Committee on Large Dams, 37 pages, dated 1983. General guidelines for inspections.

2. District Form # FC 1166 (05-13-93) Preliminary Post Earthquake and Semiannual Inspection, Dams and Reservoirs, 1 page. (green)

3. District Form # FC 1167 (05-13-93) Detailed Post Earthquake Inspection Report, Dams and Reservoirs, 2 pages. (white)

4. District Form # FC 1049 (03-26-91) Preliminary Post Earthquake and Semiannual Inspection Form, Pump Station, 1 page. (pink)

5. District Form # FC 1050 (03-26-91) Detailed Post Earthquake Inspection Report, Pump Stations, 2 pages.(off-white)

6. District Form # FC 1051 (03-26-91) Detailed Post Earthquake Inspection Report, Pipelines, 2 pages. (yellow)

7. How Earthquakes are Measured, CDMG Notes Number 23, California Division of Mines and Geology, 1 page, double sides.

8. Dam Inspection Package, by Tepel and Sutcliff, dated June 1988, 5 pages, District Document # 45-1270tp.

9. Santa Clara Valley Water District Preliminary Post Earthquake and semiannual Inspection form, Dams and Reservoirs, dated June 1988, 4 pages, District Document # 45-1270a

10. Package of information for Almaden, Anderson, Calero, Coyote, Guadalupe, Lexington, Stevens Creek, and Vasona Dams, dated approximately 1988. For each dam there is a short check list, a reservoir area-capacity table and a general plan map of the dam. Calero has two general plans, one for the main dam and one for the auxiliary dam. Coyote has a second plan showing the inlet structure, the outlet structure, and the spillway.

5.0 Definitions And Terminology

Abutment: the native sloping valley wall, usually bedrock, against which the embankment was placed.

Accelerometer: A seismograph for measuring ground acceleration as a function of time.

Active fault: A fault along which slip has occurred in historical (or Holocene, last 11,000 yr) time or earthquake foci are located.

Axis: a line along the crest of a dam from one side to another. Usually, as at Lenihan Dam, in the center of the crest.

Creep (slow fault slip): Slow slip occurring along a fault, without producing earthquakes.

Downstream: the spillway side of the dam.

Earthquake: The vibrations of the earth caused by the passage of seismic waves radiating from some source of elastic energy.

Epicenter: Directly above the focus, on the Earth's surface, is the epicenter. Earthquake waves reach the epicenter first. During an earthquake, the most violent shaking is found at the epicenter.

Focus: The point beneath the Earth's surface where the rocks break and move is called the focus of the earthquake. The focus is the underground point of origin of an earthquake.

Foreshocks: Smaller earthquakes preceding the largest earthquake of a series concentrated in a restricted crustal volume.

Groin: the line where the embankment fill meets original ground on the left and right abutment.

Intensity (of earthquakes): A measure of ground shaking obtained from damage done to structures built by humans, changes in the Earth's surface, and felt reports.

Liquefaction (of soil): Process of soil and sand behaving like a dense fluid rather than a wet solid mass during an earthquake.

Love waves: Seismic surface waves with only horizontal shear motion transverse to the direction of propagation.

Magnitude (of earthquakes): A measure of earthquake size, determined by taking the common logarithm (base 10) of the largest ground motion recorded during arrival of a seismic wave type and applying a standard correction for distance to the epicenter. Three common types of magnitude are Richter (or local) (M_L), P body wave (m_b), and surface wave (M_s).

Normal fault: A dip slip fault in which the rock above the fault plane has moved downward relative to the rock below.

Oblique-slip fault: A fault that combines some strike-slip motion with dip slip motion.

P wave: The primary or fastest wave traveling away from a seismic event through the rock and consisting of a train of compressions and dilatations of the material.

Rayleigh waves: Seismic surface waves with ground motion only in vertical plane containing the direction of propagation of the waves.

Right and left: always as perceived by a person standing on the crest of the dam, with the reservoir at your back, and looking downstream.

Right-lateral fault: A strike-slip fault on which the displacement of the far block is to the right when viewed from either side.

Seiche Oscillations: (standing waves) of the water in a bay or lake.

Seismic wave: An elastic wave in the Earth usually generated by an earthquake source or explosion.

Seismograph: An instrument for recording as a function of time the motions of the Earth's surface that are caused by seismic waves.

Seismometer: The sensor part of the seismograph, usually a suspended pendulum.

Slip (fault): The relative motion of one face of a fault relative to the other.

Strain (elastic): The geometrical deformation or change in shape of a body. The change in an angle, length, area, or volume divided by the original value.

Stress (elastic): A measure of the forces acting on a body in units of force per unit area.

Stress (drop): The sudden reduction of stress across the fault plane during rupture.

Strike-slip fault: A fault whose relative displacement is purely horizontal.

Strong ground motion: The shaking of the ground near an earthquake source made up of large-amplitude seismic waves of various types. Typically defined as the duration from the first to the last occurrence of shaking greater than 0.05g.

Surface waves (of earthquakes): Seismic waves that follow the earth's surface only, with a speed less than that of S waves. There are two types of surface waves – Rayleigh waves and Love waves.

Swarms (of earthquakes): A series of earthquakes in the same locality, no one earthquake being of outstanding size.

S wave: The secondary seismic wave, traveling more slowly than the P wave and consisting of elastic vibrations transverse to the direction of travel. It cannot propagate in a liquid.

Thrust fault: A dip-slip fault in which the upper rocks above the fault plane move up and over the lower rocks, so that older strata are placed over younger.

Upstream: the reservoir side of the dam.

6.0 References

Performance of earth dams during the Loma Prieta earthquake, by L. F. Harder, Jr., California Department of Water Resources; R. L. Volpe, R. L. Volpe & Associates; J. D. Bray, University of California; and R. V. Rodda, Wahler Associates. p. D3-D26, 1999

Other References:

Dam Inspection Package, SCVWD

Guide to Post-Earthquake Investigations of Lifelines (ASCE).

Guidelines for Inspection of Dams Following Earthquakes (US Committee for Large Dams)

Post-Earthquake Investigation Field Guide, Learning from Earthquakes (Earthquake Engineering Research Institute)

APPENDIX E

GEOTECHNICAL CONSIDERATIONS FOR EARTH DAMS

Appendix E

GEOTECHNICAL CONSIDERATIONS FOR EARTH DAMS

1.0 Introduction

This chapter presents some basic geotechnical engineering concepts related to the interpretation of instrumentation in earth dams. A basic familiarity with these concepts is necessary to understand the performance of various instruments and how to analyze the instrumentation data. The geotechnical concepts discussed are related to pore pressure, deformation, seepage and the mobilization of shear strength. Specific attention is drawn to the condition of water under static conditions and water flowing through earth structures. For a more detailed and rigorous discussion of the geotechnical engineering considerations discussed herein, the interested reader is referred to any of the classic geotechnical textbooks and manuals that deal with these topics (e.g., Terzhagi and Peck, 1967; Lambe & Whitman, 1967; Cedergren, 1975; and Corps of Engineers EM 1110-2-1908, Volpe & Kelly, 1985).

2.0 Basics of Soil Structure, Stress And Pore Pressure

2.1 Soil structure

Soil is considered to exist as a three-phase system consisting of mineral particles, air, and water. The mineral particles consist of solid particles between which are spaces called voids or pore spaces. The pore spaces are typically filled with air and/or water. When all the voids are filled with water, the soil is said to be saturated. If any air (gas) is present in the pore spaces, the soil is said to be unsaturated. The term partially saturated or vadose zone is used by soil scientists when referring to unsaturated soils that overly the groundwater table. A diagram showing the weight and volume relationship for the three phases of a soil is shown in Figure E-1, along with corresponding definitions for the various weights and volumes.

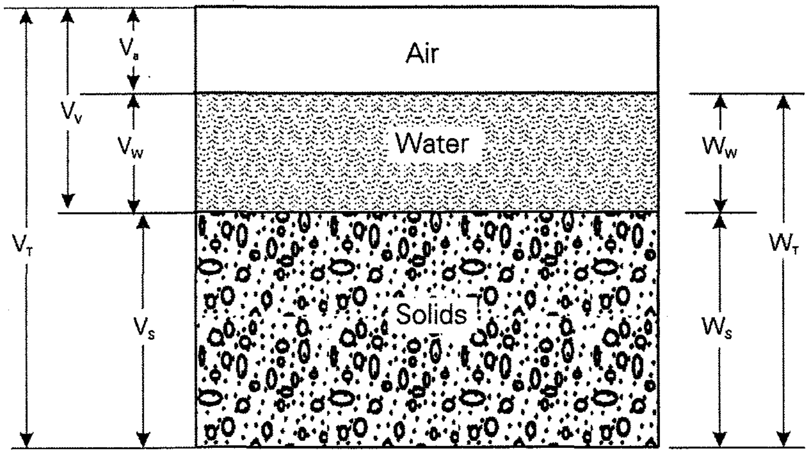


Figure E-1 - Phase Relationship of Soil

Based on the three phase relationship of soils shown in Figure E-1, the following relationships and definitions apply.

Volumes

Volume of Air = $\mathbf{V_a}$; Volume of Water = $\mathbf{V_w}$; Volume of Solids = $\mathbf{V_s}$
Volume of Voids = $\mathbf{V_v}$ = Volume of Air + Volume of Water = $\mathbf{V_a + V_w}$
Total Volume = $\mathbf{V_t}$ = Volume of Voids + Volume of Solids = $\mathbf{V_v + V_s}$

Weights

Weight of Air ≅ 0; Weight of Water = $\mathbf{W_w}$; Weight of Solids = $\mathbf{W_s}$
Total Weight = $\mathbf{W_t}$ = Weight of Water + Weight of Solids = $\mathbf{W_w + W_s}$
Water Content = w =Weight of Water / Dry Weight of Solids = $\mathbf{W_w / (W_t - W_w)}$

Definitions

Void Ratio = e = Volume of Voids / Volume of Solids = $\mathbf{V_v / V_s}$ expressed as decimal
Porosity = n = Volume of Voids / Total Volume = $\mathbf{V_v / V_t}$ expressed in percentage
Porosity= $\mathbf{n = e / (1+e)}$
Total Unit Weight = $\gamma_t = \mathbf{W_t / V_t}$; Dry Unit Weight = $\gamma_d = \gamma_t \mathbf{/ (1+}w\mathbf{)}$
Specific gravity of solids = $\mathbf{G_s}$
Unit weight of water = γ_w = 62.4 lb/cu. ft.
Void Ratio = e = $(G_s * \gamma_w / \gamma_d) - 1$
Saturation = $\mathbf{S = w*G_s / e}$ expressed in percentage

2.2 Soil Types

Soils are categorized into two broad groups: *cohesionless* soil and *cohesive* soil. Cohesionless soils are those soils that consist of particles of rocks or minerals having individual grains usually visible to the eye. These materials consist of sands and gravels, are granular and nonplastic, meaning that they do not stick together when unconfined, and are distinguished by the fact that they have little or no strength when air dried. Usually these soils have not been altered by chemical decomposition. Inorganic silt is fine grained with little or no plasticity and, although fine-grained is considered a cohesionless material. Cohesive soils such as clays consist of fine-grained materials, with microscopic and submicroscopic particles resulting from chemical decomposition of rocks. These materials have some strength when unconfined and air dried. Water affects the interaction between the mineral grains and gives plasticity or cohesion between the particles. Many cohesive soils have been affected by previous geologic loads (e.g., glacier, previous overburden, etc.)

In geotechnical engineering, soils are identified using the Unified Soil Classification System (USCS) as defined by American Society for Testing and Materials (ASTM) Test Method D-2487. Using this descriptive method, the classification system is simplified to a two letter designation: the first defines the major soil type; and the second letter describes either the gradation characteristics for cohesionless materials or the plasticity for cohesive soils. The letters used for the four major soil types, along with their corresponding range in grain sizes, are as follows:

- **G** for Gravel (10 mm-75 mm);
- **S** for Sand (0.075 mm-10 mm);
- **M** for Silt (0.002 mm-0.075 mm); and
- **C** for Clay (<0.002 mm).

For cohesionless soils, the second letter of the USCS designates whether the gradation is well graded (W) or poorly graded (P). For cohesive soils, the second letter designates whether the fines are of low plasticity (L) or high plasticity (H). For cohesionless soils with more than 12 percent passing the No. 200 sieve (0.075mm), the first letter designates whether the material is sand or gravel, and the second letter designates whether the fines are silty or clayey. Some USCS designations for typical soils include SW for a well graded sand, SM for a silty sand, GP for a poorly graded gravel, GC for clayey gravel, CH for clay of high plasticity, and ML for silt of low plasticity. Organic (O) soils are also covered in the USCS, but are only encountered under special geologic conditions. An accurate use of the USCS is highly recommended when describing soils for geotechnical engineering purposes because reasonably accurate engineering properties of shear strength, permeability, compressibility, and potential construction difficulty can be estimated based solely on the USCS symbol.

2.3 Concept of Effective Stress, Pore Pressure and Groundwater Level

In geotechnical engineering, the terms stress and pressure are defined as force per unit area, with typical units of pounds per square inch (lb/in^2) or pascals (Pa). In the practice of geotechnical engineering, pressure and stress are sometimes used inter-changeably. The total force applied to or acting within a given area is known as the total stress. That component of the total stress that is transmitted by grain-to-grain contact within the soil mass is called the effective stress. The component of the total stress transmitted through the pore water is called the pore water pressure, or the neutral stress. The total stress consists of the sum of the effective stress and the pore water pressure. This relationship is known as Terzaghi's principal of effective stress and is expressed mathematically as follows:

$$\sigma = \sigma' + \mu$$

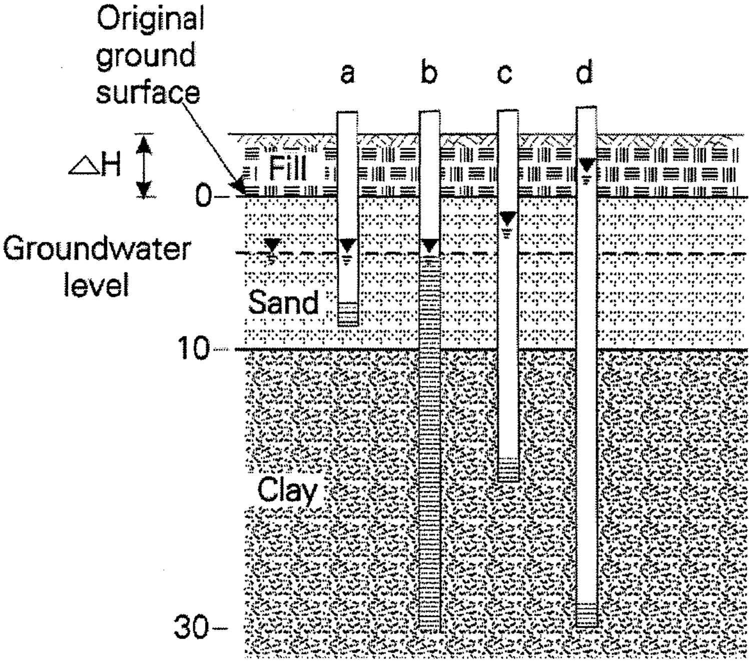
where:

σ = total stress
σ' = effective stress
μ = pore water pressure

It should be noted that the total stress is calculated by measuring the total density of the material in the field or in the laboratory. The total stress at any depth is the sum of the appropriate total unit weight times the layer thickness. In a similar manner, the pore pressure acting within a soil mass can be determined by using an open well or other type of piezometer, as discussed later in this section. The effective stress is then determined as the difference between the total stress and the pore pressure. In cohesionless soils, it is generally assumed that the pore pressure acting above the water table is zero and that the effective stress is equal to the total stress. In certain soils, especially clayey soils with high suction values, the previous assumption can lead to serious error.

Only the total stress and the pore pressure can be physically measured in the field. The total stress is simply defined as the total weight of soil plus water acting at a given depth (plus any additional surcharge loads) and is determined by measuring the total unit weight (γ_t) of the soil.

The pore pressure is measured by a piezometer that measures the interstitial pore fluid pressure. It is important to note the possible difference between pore water pressure and the ground water level. The groundwater level is defined as the upper water surface at which the pressure is atmospheric. Pore pressure is a measure of the interstitial water pressure and may or may not be related to the groundwater level.



E-2 - Concept of effective stress and pore water pressure

Consider, for example, the condition shown in Figure E-2 that depicts four pipes installed in a soil within which there is no flow of groundwater, although a fill of thickness △H has recently been placed. All pipes, except pipe (b), are solid except they are perforated near the bottom 18-inches. Pipe (b) is perforated throughout its length. The soil strata are comprised of an upper layer of sand that is about 10 feet thick, underlain by about 40 feet of clay and then bedrock (the entire column is not shown). Under the static groundwater conditions shown in Figure E-2, the water pressure increases uniformly with depth below the groundwater level which was about 4 feet deep below the original ground surface before the fill was placed. When such a condition exists, the level of water with all of the pipes will rise to the groundwater level, independent of the locations of the perforations or the type of soil present. If we were to monitor the pipes sometime after the fill has been placed, but before the consolidation of the clay has occurred, we may find the water level conditions shown in Figure E-2. As shown, standpipes (a) and (b) show the same water level as before the placement of the fill. This is the static groundwater level. The water level in pipes (c) and (d), however, are higher that the static groundwater level. The reason for this apparent anomaly can be explained using the concept of effective stress.

As discussed above, the total stress is comprised of the effective stress and pore pressure increment, or:

$$\Delta\sigma = \Delta\sigma' + \Delta\mu$$

where:

$\Delta\sigma$ = total stress increment

$\Delta\sigma'$ = effective stress increment; and,

$\Delta\mu$ = pore pressure increment

For the situation shown in Figure E-2, and assuming a total unit weight of the added fill equal to 120 b./cu. ft., the total stress increment imposed by the fill is equal to 1,200 lb./sq. ft. (psf) (i.e., 10*120). Because of the much higher permeability of the sand material relative to the clay material, the total stress is almost immediately converted to effective stress. In other words, the excess pore pressure within the sand layer caused by the fill placement is almost immediately dissipated and, likewise, the effective stress within the sand is increased by 1,200 psf. This is reflected in the response of the water levels within pipe (a) in the sand material. Since the added fill did nothing to change the groundwater level, this pipe still reflects the same level of the static groundwater level as before the fill was placed.

This is not true in the clay layer. Pipes (c) (located at the quarter-thickness of the clay) and pipe (d) (located at the mid-height of the clay) reflect a water levels that are higher than the static groundwater level. When the fill was placed, the total stress was increased 1,200 psf which is equivalent to approximately 19.2 feet of water. This would have been the increase in total pore pressure acting throughout the entire thickness of the clay layer (due to the areal extent of the fill) immediately following placement of the fill. When reviewing the total pore pressure at any point, that portion that is greater than that imposed by the static water level is referred to as excess pore pressure. Only the clay layer is said to have excess pore pressure. The excess pore pressures developed within the clay by the placement of the fill will slowly dissipate. The rate of dissipation is governed by the consolidation characteristics of the clay. The dissipation of excess pore pressures is likewise accompanied by an upward or downward flow of water out of the clay layer as the consolidation occurs. Using the concept of effective stress, at any moment during the consolidation process, the amount of increase in the effective stress will exactly equal to difference in total stress increase and the amount of excess pore pressure dissipated. In other words:

$$\Delta\sigma' = \Delta\sigma - \Delta\mu$$

Due to the thickness of the clay, it is intuitively obvious that it takes longer for the excess pore pressure in the middle of the clay to dissipate than near the boundary with the sand. This is depicted in Figure E-2. Pipes (c) and (d) indicate the pore pressure in the clay at levels (1) and (2). The water level in pipe (c) is shown lower than in pipe (d) because it is closer to the sand interface (which acts as a drainage face) and more dissipation of pore pressure has occurred at level (1) than at level (2). Immediately following placement of the fill, the water levels in pipes (c) and (d) would have been very nearly equal to 19.2 feet higher than the static groundwater level. With time, as complete consolidation occurs and all excess pore pressure has dissipated, the water levels in pipes (c) and (d) will return to the same level as pipe (a) and (b), or the static groundwater level prior to the placement of the fill.

In the case illustrated in Figure E-2, although pipe (b) is bottomed in the clay, since it has perforations throughout its length, it reflects the static groundwater level and totally misses the excess pore pressures acting within the clay. The reason for this is a result of the difference in permeability; the permeability of the sand is much greater than of the clay. Consequently, excess pore pressures in the clay will cause an upward flow of water from the clay to the sand during the consolidation process via the pipe. However, since the pipe is perforated throughout its length, the excess water quickly is consumed in the static water column such that no excess pore pressure would be detected. In the more general case of a perforated pipe installed through two or more strata, either with perforations throughout or surrounded with sand throughout, an undesirable vertical connection between strata is created, and the water level in the pipe will be misleading. Pipe (b) is referred to as an observation well. Pipes (a), (c), and (d), which measure the standing water at a given depth, are referred to as piezometers.

The remainder of this chapter will focus on piezometers, pore pressure measurements, and seepage analyses, the principal analytical measurements and analyses associated with earth dams. A brief definition of selected geotechnical terms is presented in Table E-1.

Table E -1 Definition of Selected Geotechnical Terms

Term	Definition
Piezometer	An instrument used to measure water pressure within a soil.
Soil	Made up of virtually any combination of the four basic soil types of gravel, sand silt, and clay.
Pore pressure	In a saturated soils, the water pressure acting within the voids of the soil is defined as the pore pressure.
Total stress	The total stress acting at a given location within a soil is equal to the combined effect of the total unit weight of the soil material plus the interstitial pore water pressure.
Effective Stress	The average interstitial, grain-to-grain, contact stress that controls the shear strength of a given soil; by definition, the effective stress is equal to the total stress minus the pore pressure.
Phreatic Surface	An imaginary surface that defines the boundary of complete saturation within a soil; above the phreatic surface we assume the pore pressure is equal to zero, while below the phreatic surface the pore pressure is defined by the flow conditions existing within the soil.
Equipotential	A line drawn on a flow net that connects points of equal total head.
Flow Line	A line drawn on a flow net that depicts the direction or path of flow

3.0 Piezometers and Pore Pressure

3.1 Basic Considerations

Piezometers are installed in order to assess the piezometric response of a dam, including both the embankment and its foundation, under different reservoir loading conditions. These instruments are installed either during or after construction. If they are installed during construction, you are virtually unlimited as to the type of instrument to be used, or where to locate them; if they are installed after construction, however, as is the case for most District dams, you are then somewhat limited as to the type of instrument, and its location, since in order to install a piezometer, you must first drill a hole. One can not always control the reservoir condition when the field work is done. If the reservoir is full, you may be limited to installing all piezometers within the downstream half of the dam. Nevertheless, it is not until after the instrument has been installed that the work really begins. This includes the effort to read, maintain, and interpret the instrument readings.

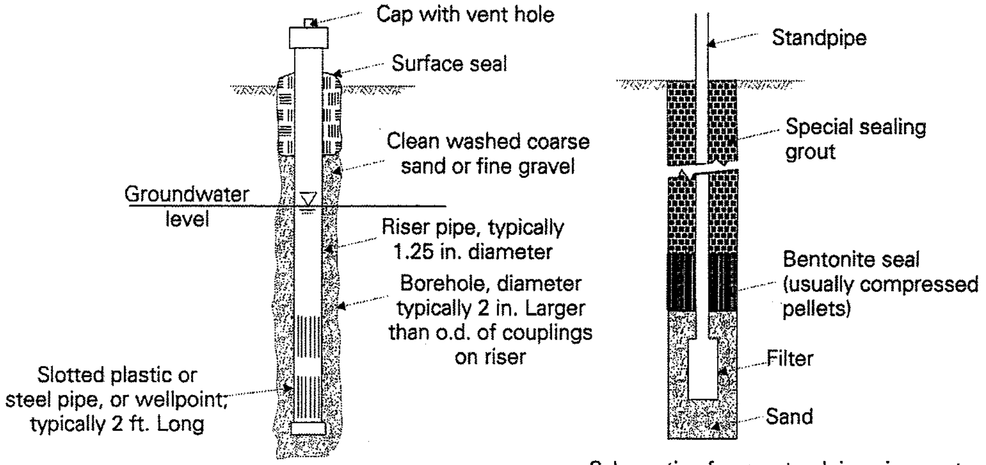
3.2 Basic Types of Piezometers

The District has three basic types of piezometers; open well, pneumatic, and vibrating gauge (electronic). These three types of instruments, along with their inherent differences and difficulties, are described below.

3.2.1 Open Well Piezometers

The open well piezometer, as its name implies, is simply an open well (i.e., vented to the atmosphere) which measures a prevailing water pressure affecting the instrument. This could be the highest or lowest pore pressure acting over the height of the instrument, and is completely dependent on the type of open well piezometer used to measure the pore pressure and, to a lesser degree, the permeability of the soil surrounding the tip of the piezometer. This is further explained below.

There are two types of open well piezometers. The simplest, and perhaps most misunderstood, of this type consists of a small diameter (typically ½"-1") PVC pipe that has openings (e.g., hack saw cuts at ½"-1" center-to-center spacing) over a portion of the lower section of the piezometer through which water can flow into the pipe. This type of instrument is customarily referred to as an observation well. The second of these consists of a similar installation; however, only a limited section, generally the bottom 18", of the pipe is slotted which, in turn, is connected to an unslotted pipe which is carried up to ground surface. A section above the slotted part is sealed off using a bentonite seal to prevent surface water or other horizons from directly affecting the instrument. These two types of instruments are presented in Figure E-3.


Cap with vent hole
Surface seal
Clean washed coarse sand or fine gravel
Groundwater level
Riser pipe, typically 1.25 in. diameter
Borehole, diameter typically 2 in. Larger than o.d. of couplings on riser
Slotted plastic or steel pipe, or wellpoint, typically 2 ft. Long
Standpipe
Special sealing grout
Bentonite seal (usually compressed pellets)
Filter
Sand

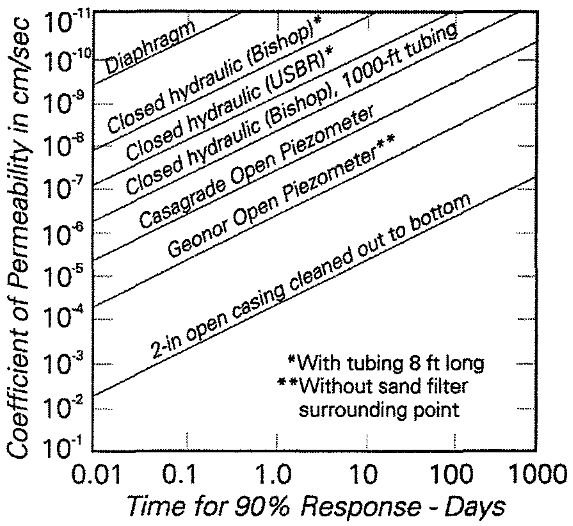
Schematic of observation well

Schematic of open-standpipe piezometer installed in a borehole

Figure E-3 - Schematic of Observation Well and Open Standpipe Piezometers

If one were to place these two types of open well piezometers at the bottom of a 10-ft. deep swimming pool, how high up would the water rise inside the riser pipe? In both open well piezometers, the water would rise up to the surface of the swimming pool and each piezometer would be said to be reading 10 feet of head (i.e. 10*62.4 = 624 psf). This represents the free standing water surface in this example. If we were able to take a slow motion picture of the two instruments immediately after installation, however, we would see how different the two instruments react to water pressure. In the open slotted pipe, water is entering through every opening impacted by water pressure and running down the pipe until it hits the water level created by the underlying slot. This process continues until the tube is filled and no more water wants to enter the pipe. For the open well piezometer with only a bottom 18-inch section that is slotted, the water pressure acting along the sides of the instrument continues to force water to flow into the pipe until the water pressure both inside and outside the pipe is balanced. If the inner tube is 1/2-inch diameter, and the pool is 10 feet deep, then a total of about 24 cubic inches of water would have flowed into the inner pipe. This is the reason for the hydrodynamic lag discussed later in this section, and specifically why we do not like to use open well piezometers for measuring pore pressure in semi-impervious to impervious soils.

Suppose that either of the piezometer types had been placed at a depth 2 feet, 5 feet, or 7 feet below the pool surface. It is obvious that in each instance, the water would rise to the pool surface, and the piezometers would be said to be reading 2, 5, or 7 feet of head. Now, let us assume that there is an inflatable (impermeable) membrane at a depth of 5 feet. Let us further assume that the imaginary membrane was inflated with an equivalent pressure equal to 10 feet of water (624 psf or 4.33 psi), and that the membrane is unaffected by the water above it. If we now place two of our tip piezometers in the pool, one just above and one just below the membrane, the one just above the membrane would read 5 feet of head and the one just below would read 10 feet of head. Notice that although the piezometers are only inches apart, the difference in the readings is 5 feet of head. For this example, the upper 5 feet of the pool is said to be under hydrostatic water pressure conditions (commonly referred to as hydrostatic conditions) and the lower section of the pool (i.e., that section beneath the inflatable membrane) is said to be under excess hydrostatic pressure. In this example, it is under 5 feet of excess hydrostatic pressure. Note that regardless of what level beneath the membrane that we placed our tip piezometer (e.g., 6, 7, 8, 9 or 10 feet), the piezometer would have recorded 10 feet of head. Also, when we compare this example with the previous example of hydrostatic conditions, we note that at a depth of 5 feet we had an excess hydrostatic head of 5 feet, at 6 feet we had an excess hydrostatic head of 4 feet, and so forth down to 10 feet which had no excess hydrostatic head.



*With tubing 8 ft long
**Without sand filter surrounding point

Figure E-4 - Approximate response time for various types of piezometers (Terzhagi and Peck, 1967)

Most piezometers require some movement of pore water into the device to activate the measuring mechanism. When pore water pressures change, the time required for water to flow into the piezometer to create equalization is called the hydrostatic time lag. The hydrostatic time lag is dependent primarily on the permeability of the soil, type and dimensions of the piezometer, and the change in pore water pressures. The volume of flow required for pressure equalization at a diaphragm piezometer is extremely small, and the hydrostatic time lag is very short. For an open standpipe piezometer, the time lag may be reduced by providing a large intake area and reducing the diameter of the riser pipe, thereby reducing flow required for pressure equalization. The pressure of gas bubbles (natural or result of corrosion) can also cause a time lag. Hydrostatic time lag is not significant when piezometers are installed in highly pervious soils such as coarse sands. Those piezometers that have a long time lag are described as having a slow response time. The time required for establishing equalization of pore water pressures after installing or flushing a piezometer is called the stress adjustment time lag, sometimes referred to as the installation time lag or hydrodynamic time lag. An estimate of the hydrodynamic time lag aids in selecting the proper type of piezometer for given subsurface conditions at a given site. The order of magnitude of the time required for 90% response of several types of piezometers installed in homogeneous soils can be theoretically determined. As stated in Dunnicliff (1988), the 90% response is considered reasonable for most practical purposes since the 100% response time is infinite. The approximate response times for different piezometers are shown in Figure E-4.

The response time of open standpipe piezometers can be estimated from equations derived by Penman (1960). As an example,

$$t = 3.3 \times 10^{-6} \frac{d^2 \ln [L/D + (1\ (L/D)^2)^{1/2}]}{kL}$$

where;

t = time required for 90% response, days
d = inside diameter of standpipe, centimeters (cm)
L = length of intake filter (or sand zone around the filter), centimeters (cm)
D = diameter of intake filter (or sand zone), centimeters (cm)
k = permeability of soil, centimeters per second (cm/sec)

Similar equations and other procedures have been developed by Hvorslev (1949) and others. It is often more desirable to measure time lag in the field by comparing reservoir fluctuation with desired piezometer readings as illustrated in Figure E-5.

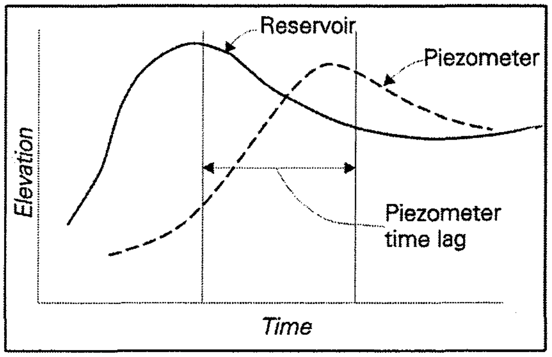


Figure E-5 - Time lag measurement from from time-history plot of reservoir level and piezometer

When measuring an open well piezometer, our goal is to determine the elevation of the water surface acting within the riser pipe. This is customarily accomplished by inserting a moisture sensitive resistor (an M-scope) through the top of the riser pipe and carefully lowering it down the pipe until the reading indicates that the resistor has encountered water. The elevation of the water surface is then computed by subtracting the amount of stickup of the riser pipe above the adjacent ground surface from the total depth to the water surface, and then subtracting the resulting depth from the ground surface. The following is a hypothetical example:

Measured depth to water surface	= 24.5 feet
Stick up above ground surface	= 2.5 feet
Ground surface	= 105.0 feet
Elevation of water surface	= [105 - (24.5-2.5)] = 83.0 feet

3.2.2 Pneumatic Piezometers

There are several types of pneumatic piezometers. In general, those operated by the District have been installed in drill holes at various dams. The piezometers are made of either porous stone or ceramic material and are about 8-10 inches in length. They are connected to the ground surface by one or more small diameter flexible tubes that are, in turn, encased in a hard (but flexible) butalic rubber. The way in which a newer pneumatic piezometer functions is as follows. Water (under pressure equal to the pore pressure) passes through the tip material and enters a hermetically sealed portion of the piezometer tip. It then comes in contact with a flexible rubber membrane inside the tip. The rubber membrane expands to close an air passageway through the piezometer. When a reading is taken, air pressure is introduced at ground surface and travels down the tube where it encounters the closed passageway. The air pressure at the ground surface (provided by a charged gas bottle) is slowly increased and monitored until a sufficient pressure is exerted to close the membrane, thereby allowing air to pass through the tip and return to ground surface. The pore pressure is then computed by noting the pressure required to open the rubber membrane, and then subtracting a manufacture-determined constant which, in essence, is equal to the friction losses to compensate for the gas flowing through the small diameter tubes, and the extra pressure required to force the membrane to move. A typical installation of a pneumatic piezometer installed in a boring is shown in Figure E-6.

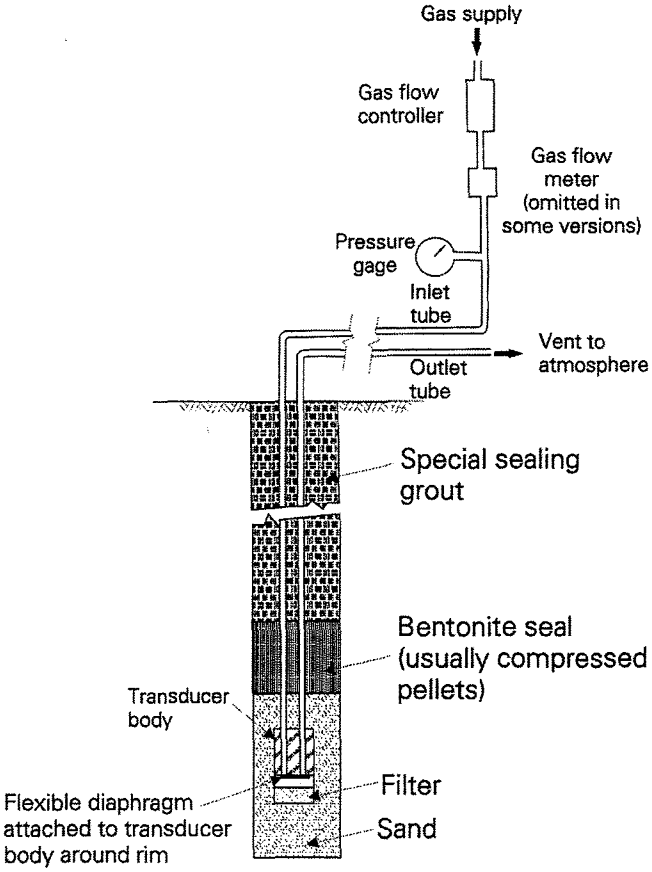

Gas supply
Gas flow controller
Gas flow meter (omitted in some versions)
Pressure gage
Inlet tube
Vent to atmosphere
Outlet tube
Special sealing grout
Bentonite seal (usually compressed pellets)
Transducer body
Flexible diaphragm attached to transducer body around rim
Filter
Sand

Figure E-6 - Schematic of pneumatic piezometer installed in a boring

Note that the volume of flow required to initiate a reading in a pneumatic piezometer is constant (i.e., independent of the pore pressure) and is equal to only a small fraction of what is required for an open well piezometer. Since the velocity or speed with which water can flow through a soil is directly dependent on its coefficient of permeability, then the amount of time required for an open well piezometer to respond to a new pore pressure is directly related to how long it takes for the incremental volume of water to flow into (or out of) the open well piezometer, which in turn is directly related the average soil conditions around the tip. Another way of stating this important concept is that geologic conditions and soil types can impact the readings measured by piezometers. This important concept is further discussed in the following section.

For reference purposes, we generally refer to the pore pressure in terms of feet of head. For completeness, however, we should define what this really means in terms of pressure. The unit weight of water is equal to 62.4 pounds per cubic foot (pcf). If we had a column of water 1.0 foot high, it would exert a pressure at the base equal to 62.4 pounds per square foot (psf). Gauge pressures used to measure piezometers are typically read in pounds per square inch. Since there is 144 square inches in 1.0 square foot, then 1.0 pound per square inch (psi) is equal to 2.308 (144/62.4) feet of head.

Finally, it should be noted that pneumatic piezometers are very time consuming to read and fraught with problems and concerns. Although the District should continue to read all existing units, they have plans to gradually get rid of the pneumatic piezometers once the readings for new piezometers have been verified.

3.2.3 Vibrating Wire Piezometers

The vibrating wire piezometer has a metallic diaphragm separating the pore water from the measuring instrument. As shown in Figure E-7, a tensioned wire is attached to the midpoint of the diaphragm such that the deflection of the diaphragm causes a change in wire tension, and pore pressure measurements are made by exciting and measuring the frequency of the response. The District has used two installation techniques for vibrating wire piezometers at Lenihan Dam. The first method, and more conventional, was the down-hole method. The installation technique for this approach is to first clean the hole as necessary and tremmie sand in the hole until the desired depth of the piezometer has been reached. The piezometer is inserted inside a cloth sack filled with sand and the bag is the tied off to a rigid piece of plastic pipe. As the instrument is lowered down the drill hole, the exiting cable from the piezometer is then tied securely to the rigid pipe. This is repeated as often as necessary until the piezometer reaches the desired depth. The hole is then grouted with cement-bentonite slurry, the consistency of which matches the approximate rigidity of the native soil. The second method of installation is to push the piezometer (heavy duty stainless steel tip) with a Cone Penetrometer Truck. This method is very fast and reliable, but is not yet accepted by DSOD. It is necessary to protect the tip from overstressing during installation by using a protective shield. The hole is again grouted with a cement-bentonite slurry as the mandrel that was used to push the piezometer tip into place is removed. This method is also limited to relatively flat ground for the heavy CPT Truck, such as the dam crest, or on access roads built out to the area of interest. The CPT method was used to install four vibrating gauge piezometers in the U/S face at Lenihan Dam to depths of about 70 feet with no real problems. These piezometers have continued to respond identically to the conventionally placed piezometers since they were installed in the summer of 2000.

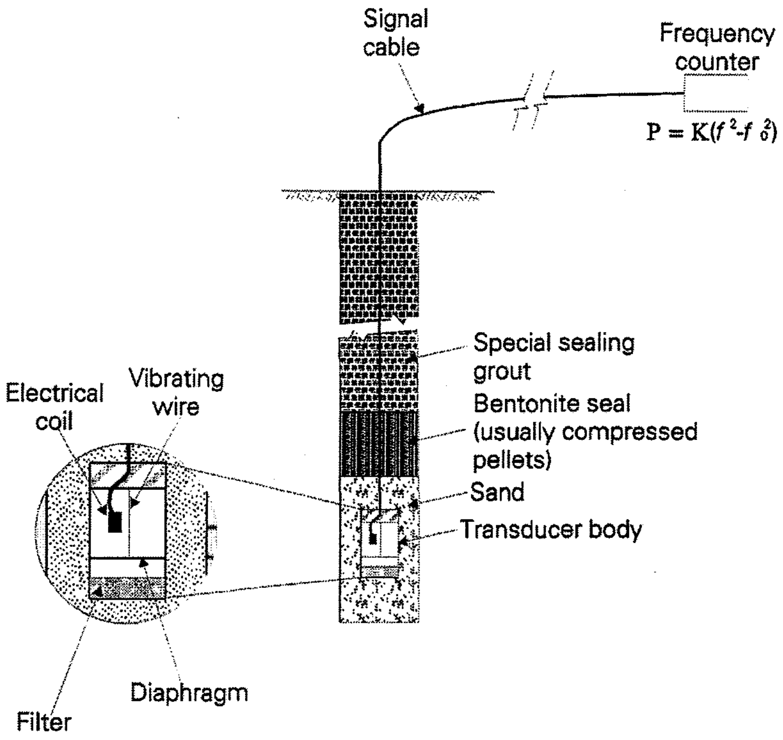


Figure E-7 - Schematic of vibrating wire piezometer installed in a boring

4.0 Seepage Examples

Prior to launching into a study of the piezometric response in earth dams it is important to discuss certain fundamentals of fluid flow in soils which is referred to as seepage analysis.

4.1 Flow Net for One Dimensional Flow

Figure E-8a (see end of Appendix) shows a tube 4 ft x 4 ft in cross section by 16 ft high through which steady state vertical flow is occurring. The tube of homogeneous soil is 10-ft high, has a porosity of 33%, and a permeability of 0.1 ft/min and water is flowing vertically downward. Atmospheric pressure is maintained at the top of the reservoir water (elev. 16). Datum is conveniently taken at the tail water (elev. 0). The values of total head, elevation head, and pressure head are also plotted on Figure E-8b. Based on the dimensions of flow shown in Figure E-8a, and the variation in total head shown in Figure E-8b, then the rate of seepage through the tube, as computed by Darcy's Law, is equal to

$$Q = kiA = 0.1 \text{ ft/min} * [(16-0)/10)] * 16 \text{ sq. ft.} = 2.6 \text{ cubic ft/min}$$

where;

Q = rate of flow in units of volume per unit of time
k = the equivalent coefficient of permeability of the soil, ft/min
i = rate of change of total head per unit of length (gradient, ft/ft)
A = area through which the rate of flow is computed, sq. ft.

If we placed a drop of dye on the top of the soil (elevation 12) and traced the movement of the dye through the soil we would get a vertical flow line or a flow path. That is, each drop of water that goes through the soil follows a flow line. In the 4 ft x 4 ft tube we have an infinite number of flow lines. For convenience, five flow lines are shown on one vertical cross section in Figure E-8c; three flow lines (indicated by solid lines) are at the quarter points and one flow line is at each vertical boundary of the soil. These five flow lines divide the area into four flow channels of equal dimension, 1 ft wide. Since all flow is vertical and thus all flow lines are parallel, there is no flow from one channel into another. We can thus determine the total quantity of flow through the tube by multiplying the quantity of flow in one channel by the number of channels. In the tube in Figure E-8c, there are four flow channels per 1 ft perpendicular to the figure; thus there is a total of 16 channels.

In the sketch in Figure E-8c are also shown lines along which the total head in the flowing water is a constant. The value of total head h in Figure E-8b has been recorded at the right of each horizontal line in Figure E-8c. These horizontal lines are called *equipotential lines* since they are lines drawn through points of equal total head. Just as there was in infinite number of flow lines there are an infinite number of equipotential lines. Dividing the length of the tube with equipotential lines at equal spaces means that the total head lost between any two pair of adjacent equipotential lines is the same.

A system of flow lines and equipotential lines, as shown in Figure E-8c, constitutes a flow net. In isotropic soil the flow lines and equipotential lines intersect at right angles, meaning that the direction of flow is perpendicular to the equipotential lines and in the direction of the total head loss. The intersecting flow lines and equipotential lines form an orthogonal net. The simplest pattern of orthogonal lines is one of squares. Whereas any orthogonal pattern can be used in flow nets, the simple system of squares is the one commonly employed. From a flow net the geotechnical engineer can determine three very useful items of information: rate of flow; head; and gradient.

The first application is to see how the flow net can be used to determine the rate of flow through the soil. Consider square *A* (second square in from the left on the top row) in our flow net as noted in Figure E-8c. The rate of flow q_A through this square is

$$q_A = k * i_A * a_A$$

The total head lost in square *A* is equal to H/n_d where H is the total head lost in flow and n_d is the number of head drops in the flownet, and i_A is equal to $(H / n_d) / l$ where l is the vertical dimension of A. The cross-sectional area a_A of square *A*, as seen in plan view, is *B* as dimensioned in the figure times *L* where *L* =1 ft is the dimension perpendicular to the page.

therefore,

$$q_A = \frac{K*H*B}{n_d * l}$$

Since the shapes in the flownet are square, then $B=l$ and $q_A = k(H/ n_d)$. Since all of the flow which goes through the flow channel containing square A must pass through square *A*, the flow through square A is that for the entire flow channel. To obtain the flow per unit length L perpendicular to the page, we must multiply q_A by the number of flow channels, n_f:

$$Q/L = q_A * n_f = k * (H/n_d) * n_f$$

or

$$Q/L = kH(n_f / n_d) = kH \oint$$

The ratio n_f / n_d is a characteristic of the flow net and is independent of the permeability k and the total head loss H; it is termed the *shape factor* of the flownet and is represented by the symbol $\oint$. The dimensionless value of $\oint$ for the net in Figure E-8c is

$$\oint = n_f / nd = 4/10 = 0.4$$

and

$$q = Q/L = k \text{ x } H \text{ x } \oint = 0.1 \text{ ft/min x } 16 \text{ ft x } 0.4 = 0.64 \text{ ft}^3\text{/min/ft}$$

and

$$Q = q \text{ x } 4 = 0.64 \text{ ft}^3\text{/min/ft x } 4 = 2.6 \text{ ft}^3\text{/min}$$

It will be noted that this value of total seepage is, of course, the same that was obtained by the initial computation using Darcy's law directly.

It is possible to use the flow net to determine total head at any point. Since H is the total head lost in flow and there are ten equal head drops, H/10 (the uniform total head decrement) is lost in flow from one equipotential to the next. At the right of the flow net in Figure E-8c are shown the values of total head. It is essential to realize that the equipotentials in the flow net are drawn through points of equal *total head*, since it is the total head that controls flow. Having the total head and the elevation head for any given point, the pressure head can be readily determined. For example, consider a point in the soil at elevation 10 where the total head h = 8/10 x H=12.8 ft and h_e equals the elevation of the point, 10 ft. The pressure head is therefore equal to h- h_e =2.8 ft. The pressure head at any point is the height above the point to which water will rise in a piezometer installed at the point. Thus the water stands 2.8 ft above elevation 10 in the piezometer sketched at the left of the flow net. The pore water pressure at elevation 10 is 2.8 ft x 62.4 lb/ft^3 = 175 lb/ft^2. In similar fashion, the pressure heads at elevations 7 and 3 are 1.0 ft and (-)1.4 ft, respectively, and are shown at the left of the flow net.

Finally, the flow net can be used to determine the gradient at any point in the net. The value of gradient i for any square is equal to the total head lost in the square divided by the length of the square, $i = h/l$. Since for the flow net in Figure E-8c all of the squares are the same size, the gradient for any square is equal to h/l equal to H/(10xl) = 1.6.

Thus by the techniques described in the preceding paragraphs, a flow net can by used to determine the rate of flow, the head at any point, and gradient at any point. The example selected for this demonstration is so simple that these quantities could have been obtained easily without the flow net. The technique employed with the flow net to obtain these values holds for the most complex net. Hence the purpose of the example in Figure E-8 is to show what a flow net is and how it is used, and further, to show that the values of flow, head, and gradient are exactly correct when obtained from the flow net if the net used for the determination is itself exactly correct.

4.2 Flow Net for Two Dimensional Flow

This section discusses the use of flow nets for three situations involving two-dimensional fluid flow. The first, flow under a sheet pile wall, and the second, flow under a concrete dam, are cases of confined flow since all the boundary conditions are completely defined. The third, flow through an earth dam, is unconfined flow since the top flow line is not definitely defined (in advance of constructing the flow net).

4.2.1 Flow Under a Sheet Pile Wall

The first seepage case is an actual civil engineering problem involving a sheet pile wall: a wall to retain a building excavation, the wall around the marine terminal, the anchored bulkhead for the ship dock, etc. Figure E-9 shows a sheet pile wall driven into a silty soil having a permeability of 10^{-6} ft/min. The sheet pile wall runs for a considerable length in a direction perpendicular to the page and thus the flow underneath the sheet pile wall is considered two-dimensional.

The boundary conditions for the flow under the sheet pile wall are: *kb*, upstream equipotential; *hl*, downstream equipotential; *beh*, flow line; *mn*, flow line. Within these boundaries the flow net shown has been drawn. Having the flow net, it is possible to compute the seepage under the wall, the pore pressure at any point in the subsoil, and the gradient at any location in the subsoil. These computations are performed on Table E-2.

Seepage under the wall:

$Q/L = k \text{ x } H \text{ x } \oint = 10^{-6}$ ft/min x 25 ft x (4/8)
$= 12.5 \text{ x } 10^{-6}$ ft^3/min/ft
Exit gradient $= h/l = (25/8)/11.5 = 0.27$
Factor of Safety against piping = FS = $i_{crit.} / i_{actual} = 1/0.27 = 3.7$
Note: $i_{crit.} = (G_s - 1)/(1+e)$ For average soils, $G_s = 2.65$ and $e = 0.65$,
Therefore: $i_{crit} \cong 1.0$

A water pressure plot, such as that in Figure E-9 is useful in the structural design of the wall and in a study of the water pressure differential tending to cause leakage through the wall.

Table E-2 Flow Beneath a Sheet Pile Wall

Point	Elevation Head	Total Head	Pressure Head	Water Pressure (lb/ft^2)
a	90	90	0	0
b	60	90	30	1872
c	49	86.9	37.9	2364
d	39	83.8	44.8	2792
e	30	77.5	47.5	2964
f	39	71.3	32.3	2012
g	49	68.1	19.1	1193
h	60	65	5	312
i	65	65	0	0

The critical part of the section for possible liquefaction is where the flow near the ground surface is upward and the gradient the maximum. For the sheet pile wall this occurs at point *h*-the flow is upward, as indicated by the vertical flow lines, and the vertical gradient is a maximum here, as indicated by the fact that the square at *h* is the smallest for any square through which the flow is vertically upward. The gradient in the square next to the wall is equal to 0.27, as the computation in the example shows. Even though this gradient is considerably below that necessary to cause a quick condition (approximately 1), it is relatively high. A large factor of safety against a quick condition on the downstream portion of a structure can economically be obtained and is usually desirable because of the seriousness of a quick condition, and the fact that minor variations in soil might cause relatively large errors in the computation for exit gradient.

4.2.2 Flow Under Concrete Dam

Figure E-10 shows a concrete dam resting on an isotropic soil having a permeability of 0.1 ft/min. The section shown is actually a spillway, since water flows over the dam at certain periods. At the present time, the upstream water is at elevation 94 and the tail water at 68. The lines *AB* and *GH* represent impervious cutoff walls, usually formed by driving sheet piling into the soil. Using the same principles as were used for the example involving flow under the sheet pile wall, the seepage under the dam, the pore pressure head along the base of the dam, and the gradient in flow channel X near the downstream toe have been computed as shown in Figure E-10.

Figure E-11 shows three concrete dams resting on pervious soil. The three cases shown are identical except in case I there is no underground cutoff, case II has a sheet pile cutoff at the upstream face, and case III has a cutoff at the downstream face. The flow nets for the three cases are shown. The flow net for case I is symmetrical about the center line of the dam and the flow net for cases II and III are identical but reversed. Selected analytical results of the three cases are presented on Table E-3. For each of the three cases, the table gives the shape factor of the flow net, the quantity of underseepage, the exit gradient, and the uplift pressure head at point A. From the flow nets and the results presented on Table E-3, a comparison can readily be made among the three cases. Dams II and III have the same amount of underseepage, which is less than that for Dam I. Dam III has the lowest exit gradient, indicating that the downstream toe of the dam is the most effective cutoff location for reducing exit gradient. Dam III, however, has the greatest uplift pressure. This example shows the power of the flow net for design studies.

Table E-3 Seepage Results For Flow Beneath Concrete Dams Shown in Figure E-11

Dam	Shape Factor of Flow Net	Seepage Under Dam ($ft^3/min/ft$)	Uplift Water Pressure Head at Point A (ft)	Exit Gradient
I	4/12	6.66×10^{-3}	7.5	0.42
II	4/14	5.72×10^{-3}	7.1	0.34
III	4/14	5.72×10^{-3}	12.9	0.18

4.2.3 Flow Through An Earth Dam

As mentioned previously, the case of seepage through an earth dam is a problem of unconfined flow. In the previous two examples involving two dimensional flow, flow under the sheet pile wall and flow under the concrete gravity dams (Figs. E-9, E-10, and E-11), the boundaries containing the flow were fixed (i.e., the flow was constrained). For problems involving seepage through a dam, the flow is unconstrained. We will now look at seepage through an earth dam and assess the boundary conditions.

Figure E-12 shows a cross section of an earth dam resting on an impervious foundation. Also shown is the flow net for the steady-state seepage through the dam. The boundary conditions are defined as follows. The upstream face, which is depicted by line *AB,* is the upstream equipotential since the *total head* anywhere along this line is constant. Line *AD* represents the bottom flow line since we have assumed an impervious foundation. These two boundary conditions are definitely determined. The line *BC* is a boundary flow line and has the special characteristic that at all points on the line the pressure head is zero and is referred to as the *phreatic line*. Hence the difference in total head between two equipotentials must equal the change in elevation between the points where these equipotentials intersect the boundary flow line. In other words, *BC* is a flow line along which the total head is equal to the elevation head. The location of this top flow line is not known until the flow net is constructed. The line *CD* is neither equipotential nor flow line, but total head equals elevation head everywhere on *CD.*

Notice that in constructing the flow net, one must adhere to the requirement that equal head loss occurs between adjacent equipotential lines. For the flow net shown in Figure E-12, the graphical solution shows that the total head loss between each equipotential is equal to 1/9 of the total head loss.

If there were no rock toe in the dam shown in Figure E-12, the top flow line would exit on the downstream slope of the dam, as shown in Figure E-13a. The face *AB* would gradually erode away-the water flowing out the face will carry soil particles with it. If left unchecked, this process of internal erosion would eventually cause the entire dam to fail. In order to prevent such a failure, it is necessary to provide drains that lower the position of the top flow line. The rock toe shown in Figure E-12 is one possible form of drain; other common schemes are shown in Figure E-13b, E-13c, and E-13d. Design of a satisfactory drainage system, and control of the phreatic surface, is one of the most important problems involved in the design of an earth dam.

Even if drainage has been provided, it still is necessary to consider the stability of the entire downstream slope against a shear failure. A stability analysis is made by comparing the forces tending to cause movement (actuating forces) with those tending to resist the movement (resisting forces). One of the actuating forces acting within the downstream slope is the seepage force of water. This water force can be obtained from a flow net, as is illustrated in Figure E-12. The curved line *DE* is any assumed trial failure surface along which the factor of safety against sliding will be evaluated. This is only one of many trial failure surfaces that would be evaluated in a slope stability analysis. The water pressure head diagram along the curved line *DE* was obtained by the same procedure as was used to plot the water pressure diagram on the sheet pile wall and the uplift diagram on the concrete dam previously discussed. The computation for water pressure along the curved line can be facilitated by using the fact that the pressure head on each equipotential where it intersects the boundary flow line is zero. In other words, the water pressure at any point on an equipotential is merely the difference in elevation between the point under consideration and the point where the equipotential intersects the top flow line. This characteristic was employed to draw the pressure diagram shown in Figure E-12.

4.2.4 Flow in Nonhomogeneous and Anisotropic Soil

The preceding numerical examples considered only soil that has constant engineering properties from point-to-point vertically and horizontally - *homogeneous* soil - and soil that has similar properties at a given location on planes at all inclinations - *isotropic* soil. Unfortunately typical soils are generally nonhomogenous and anisotropic. Due to the manner in which sedimentary material is deposited, and the fact that they are built up over a period of many years, the nature of the sediments and the environment of deposition change, with the result that the soil in a deposit varies vertically and horizontally. In man-made as well as natural soil, the horizontal permeability tends to be larger than the vertical. The method of placement and compaction in earth fills is such that stratification tends to be built into the embankments. Ratios of horizontal to vertical permeability in compacted fills tend to be even larger than those in normally consolidated sedimentary clays.

In developing a flow net for nonhomogeneous and/or anisotropic soils, one must consider the difference in horizontal and vertical permeability prior to constructing the flow net. This concept is graphically presented in Figure E-14. As shown in this figure, since the horizontal permeability is generally higher than the vertical permeability, typical values may range from 2 to 100 times, the transformation is usually done on the horizontal scale. The horizontal scale must be reduced by the square root of the ratio of the horizontal to vertical permeability. That is, if the ratio of the horizontal to vertical permeability is 4, then the horizontal scale would be reduced by 2, if the ratio was 16, the transformation factor would be 4, and so forth. Also, when computing seepage volumes on a transformed flow net, note that an *effective* coefficient of permeability must be used that is equal to the square root of the product of the two permeability's.

A classic series of flow nets for a dam with anisotropic conditions are presented in Figure E-14. This example, from Buffalo Creek Coal Refuse Dam (W. A. Wahler & Associates, Inc., 1973), shows a dam with a high degree of anisotropy in the coefficient of permeability between the embankment and impervious foundation. Field and laboratory investigation results indicated that the permeability of the coarse coal waste of the embankment was about ten times greater than the effective permeability of the fine-grained foundation materials (i.e., $k_E=10k_F$). The foundation material, which had characteristics similar to a varved silt, was highly anisotropic with anisotropy ratios (k_h/k_v) of between 25:1 to 100:1. Due to the much higher permeability of the embankment compared to the foundation, note that the equipotential lines pass nearly vertically through the embankment and then fan out in both the upstream and downstream directions. Also, note that higher pore pressures are carried further under the downstream toe for the higher anisotropy ratio. Exit gradients for the seepage conditions shown in Figure E-14 indicate values of 0.27 for the anisotropy of 25:1 and a value of 0.38 for the anisotropy of 100:1.

The Buffalo Creek Coal Refuse Dam analysis is also very illustrative of the importance of seepage analysis to aid in incorporating the proper pore pressure for slope stability analyses. An enlarged cross section of the downstream toe area is presented in Figure E-15. For the slope stability analysis, it is important that the correct pore pressure be used in computing the mobilized shear strength along the hypothetical failure plane. For the assumed failure plane shown, the proper procedure to determine the pore pressure acting at points A and B for the two anisotropy ratios is presented. Notice that an imaginary equipotential line is drawn through the point in question. Once the given equipotential line intersects the phreatic surface, a horizontal line is drawn and the vertical distance between the point in question and the horizontal line determines the magnitude of the correct pressure head. Under both anisotropic conditions, note that if vertical equipotentials were assumed (i.e., the vertical distance between the point in question and the phreatic surface), which is customarily assumed to be a conservative assumption in earth dam engineering, then grossly incorrect pressure heads would have been determined.

It should be noted that properly drawn flow nets by hand, such as those in Figure E-15, are very time consuming and exasperating to develop. Today, computerized seepage analysis program that properly solve the Laplace differential equation governing seepage analyses are available. The two most widely used programs currently are SEEP2D and SEEP/W. Both are Windows based programs requiring a modest amount of effort to sort through the input data file preparation. SEEP2D is a modular component of the more familiar MODFLOW program and is not very intuitive in its application. The author has no hands-on experience with SEEP/W. Using SEEP2D, a reasonably complex seepage analysis for an earth dam can be completed in perhaps 20-30 hours by an experienced engineer. The entire hand drawn seepage analyses for the Buffalo Creek study took between 10-12 weeks of engineering effort.

The Dam Instrumentation Project will ultimately result in the detailed instrumentation of all District dams. It is highly recommended that two dimensional finite element seepage analyses be performed for all dams in order to compare actual pore pressures measured by the instruments with the results of theoretical analyses. Such analytical results will be very helpful in interpreting actual measured field data, as well as providing valuable data in the event that stability analyses are performed. When such detailed analyses were performed for Lenihan Dam, as a part of the 1998 Outlet Pipe Emergency, the results did not compare favorably with detailed instrumentation results after the piezometers had been finally installed. This disparity in fact led substantial credence to the assumption that earthquake-induced pore pressures from the 1989 Loma Prieta earthquake were still locked into the dam (RLVA, 2000).

5.0 Static Slope Stability Considerations

As suggested in the preceding section, the seepage conditions acting within a dam can have a profound impact on the stability analysis and the overall safety of the dam. If the dam has the appropriate number of piezometers in the proper locations, this will provide real data regarding pore pressures that can aid in verifying seepage analyses results. The remainder of this section provides guidance in the performance of static slope stability analysis for District dams, especially regarding the method of analysis and the appropriate shear strength for use in a particular analysis.

5.1 Laboratory Testing for Embankment Materials

A preliminary design for the instrumentation for ten District dams is presented in Appendix H. One of the principal benefits derived from installing the instruments within exploratory holes drilled through the embankment and foundation is the ability to recover undisturbed samples for laboratory testing. Due to the coarse-grained nature for most of the District dam embankments, it is necessary to use 4-inch to 6-inch diameter samples so as not to disturb the materials. Laboratory testing was performed on samples collected from Lenihan Dam during the recent instrumentation installation that focused on the downstream shell (SCVWD, 2002). The District is planning similar efforts of collecting representative samples from all dams during the DIP. This will enable the development of detailed engineering properties (strength, compressibility, and permeability) for all District dams. The engineering properties for the embankment and foundation materials can be used for all manner of future analyses. Typical results obtained for Lenihan Dam are summarized on Tables E-4, E-5 and E-6.

Table E-4
Density Test Results - Undisturbed Samples

Zone	No. of Tests	w/c %	Field Dry Unit Weight, pcf	Sat. %	Max. Dry Unit Weight, pcf	Relative Comp., %
U/S Shell	23	15.8	116.7	100	125	93
Upper Core	17	14.3	120.6	98	125	96
Lower Core	26	23.9	100.3	95	98.2	102
D/S Shell	14	13.8	122.0	95	140	87

Table E-5
Values of Coefficient of Permeability used for Engineering Analysis

Zone	Coefficient of Horizontal Permeability, k_h (cm/sec)	(ft/yr)	Coefficient of Vertical Permeability, k_v, (cm/sec)	(ft/yr)
U/S Shell	$5x10^{-8}$	0.05	$5x10^{-9}$	0.005
Upper Core	$1x10^{-8}$	0.01	$1x10^{-9}$	0.001
Lower Core	$1x10^{-8}$	0.01	$1x10^{-9}$	0.001
D/S Shell	$1x10^{-7}$	0.1	$1x10^{-7}$	0.1

Table E-6
Effective Stress-Based Shear Strengths Used in Stability Analysis

Embankment Zone	Effective Friction Angle	Effective Cohesion, tsf
Upstream Shell, Downstream Shell and Upper Core	35°	0.10
Lower Core	22°	0.31

It should be noted that there are several valid ways in which to compute the effective stress parameters used for stability purposes. These include, for example, the peak shear strength, the point of maximum obliquity (maximum occurrence of the principal effective stress ratio), or the shear strength at some arbitrary strain. It is recommended for all District dams that the method of evaluating effective stress parameters be the shear strength determined at 10 percent axial strain. The shear strength parameters thus determined will be slightly conservative, but are completely defensible and in tune with the procedure used by DSOD.

5.2 Long-term Steady State Full Reservoir Conditions

The long-term, steady state, full reservoir represents the most critical loading condition from a dam safety standpoint. In the event of a downstream slope failure, this condition would release the maximum amount of water thereby causing the greatest downstream hazard. For this analysis, it is recommended that an effective stress analysis be performed using consolidated undrained shear strength parameters. The computer program of choice for slope stability analyses at the District has been UTEXAS3, developed by Steve Wright at the University of Texas (Shinoak Software). Recently, however, a new code has emerged (SLOPE/W from Geo-Slope International Ltd.) that is more user friendly, especially under the Windows environment. Regardless of the program used for the stability analysis for long-term steady state conditions, the analysis should be performed using shear strength parameters determined using p′ vs. q plots $[(\sigma'_1 + \sigma'_3)/2$ vs. $(\sigma_1 - \sigma_3)/2]$. Rather than using the shear strength at the point of maximum stress obliquity, it is preferred to use a conservative shear strength interpretation at perhaps 5% or 10% axial strain. The minimum factor of safety for this condition is 1.5.

5.3 Stability of the Upstream Slope Under Drawdown Conditions

The loss of stability of the upstream slope under drawdown conditions is a condition that the upstream face of all District dams, except Anderson Dam because of its upstream rockfill section, undergoes periodically whenever the reservoir fills and then empties. It is a reasonable assumption that the drawdown stability for all District dams has been tested on several occasions during the approximate 50-65 year lives of the structure. Drawdown rates at most reservoirs, under full outflow conditions, vary between 0.5-1.5 feet of reservoir drawdown per day. There is a concern for drawdown stability if the average rate of reservoir drawdown can suddenly be increased (e.g. the installation of a new outlet). This is a current concern for Lenihan Dam if a new tunnel is constructed, rather than repairing the existing Outlet Pipe. A new outlet tunnel could significantly increase the reservoir drawdown rate. Preliminary instrumentation results from deep within the upstream shell suggest that the upstream shell material is very slow to begin draining once the reservoir begins its drawdown. This is not common for the upstream face to drain slowly. At Lenihan Dam, this response is undoubtedly due in-part to the very low coefficient of permeability (10^{-9} cm/sec) of the upstream shell materials. For the drawdown case, it is recommended that the shear strength used for stability purposes be that recommended by Lowe & Karafiath (1960). This method, which was proposed for evaluating the stability of dams during rapid drawdowwn, relates the undrained stress on the failure plane at failure, τ_{ff}, to the effective normal stress on the failure plane during consolidation, σ'_{fc}. This form of undrained shear strength is independent of pore pressure and should be determined for both 5% and 10% axial strain. The minimum acceptable factor of safety for the drawdown condition is 1.2.

5.4 Pseudo-Static Stability Analyses

Pseudo-static stability analyses are used to assess the stability of the upstream or downstream slope during earthquake loading. These analyses are considered appropriate for District dams since none of them have any liquefaction potential either in the dam embankment or in the foundation. It is recommended that the shear strength for use in the pseudo-static analysis is the same strength used for drawdown. Typically, results from the pseudo-static analyses are used in the simplified dynamic analysis. For the simplified method, it is necessary to determine the yield acceleration for each slope, defined as that horizontal acceleration that results in a factor of safety of 1.0. If pseudo-static results are being transmitted to DSOD, then it is also appropriate to determine the FS for a horizontal acceleration of 0.15g, since they use the FS for this acceleration value to compare with other dams under their jurisdiction.

5.5 Periodic Review of Static Stability

As part of the Dam Instrumentation Project, new samples will be obtained from the upstream and downstream slopes. It is recommended that sufficient laboratory testing be performed to define the gradation, permeability, and shear strength characteristics of the embankment materials. With these data, the District should perform new slope stability analyses to evaluate the factor of safety for the upstream and downstream slope under various loading conditions. Once these stability analyses have been performed, it is recommended that the stability of the upstream and downstream face be reviewed every 5-7 years, or more frequently if warranted, to assess new methods of analysis, changes in reservoir operation, or other changes in criteria.

6.0 Dynamic Slope Stability Considerations

Dynamic slope stability analyses are used to assess the ability of the dam to resist earthquake shaking. One of the major advancements of earth dam and geotechnical engineering following the near fatal collapse of the Lower San Fernando Dam during the San Fernando earthquake of 1972 was the incredible upsurge of research into the performance of soils during earthquake shaking. Lead by the late Professor H. Bolton Seed of the University of California at Berkeley, new analytical and laboratory methods were developed and our understanding of the performance of dams during earthquake shaking was significantly increased. Today, we understand that if a soil is not prone to liquefaction, which if it occurs can result in the near total loss of shear strength, then its performance during earthquake shaking is directly related to the intensity and duration of shaking. Initially, it was necessary to perform very labor intensive dynamic finite element analyses, and difficult laboratory test procedures, to compare the earthquake induced shear stresses with the inferred shear strength of the soil. Gradually, simplified analytical techniques were developed to assess the influence of earthquake shaking, when the soil in question is not susceptible to liquefaction.

The simplified dynamic analysis method is based on the pseudo-static analysis. In this method, it is assumed that a continuous horizontal force (thus the term pseudo-static) equivalent to some percentage of gravity is applied to the soil. A conventional stability analysis is performed with varying assumptions of the gravity component until a factor of safety equal to one is achieved. The gravitational component causing a factor of safety of one is termed the yield acceleration. Although it may not be intuitively obvious, the upstream and downstream embankment shells usually have different yield acceleration values. The essence of the simplified earthquake analysis assumes that a slope remains stable for earthquake accelerations up to the yield acceleration. If the yield acceleration is exceeded, then soil deformation will occur. In other words, the soil may deform during a brief period of earthquake shaking only to be stopped when the earthquake forces reverse themselves. Current methods of analyses allow the engineer to keep track of the minor deformations and then sum the total deformation for the entire duration of earthquake shaking.

The simplified procedure used to estimate permanent displacements involves the following steps.

1. The yield acceleration – the acceleration at which a potential sliding surface would develop a factor of safety of unity – is estimated using pseudo-static slope stability methods. First, the potential sliding surface having the minimum static factor of safety is found. Then, this potential sliding surface is subjected to various horizontal seismic coefficients to identify the yield acceleration (seismic coefficient) that results in a factor of safety of 1.0.

2. The seismic coefficient time history induced by the design seismic event within the potential sliding mass is estimated using a one-dimensional dynamic analysis. The seismic coefficient is the ratio of the force induced by an earthquake in the sliding mass to the total weight of that mass.

3. The seismic coefficient time history is compared to the yield acceleration. When the seismic coefficient exceeds the yield acceleration, downslope movement occurs along the direction of the sliding surface. Movement decelerates and stops when the seismic coefficient drops below the yield acceleration. The accumulated permanent downslope displacement is estimated by double-integrating the increments of the seismic coefficient time history that exceeds the yield acceleration. The seismic response of the embankment can be evaluated using the computer program SHAKE (Idriss and Sun, 1992) and inputting the time history of the selected design earthquake motion.

A number of the District dams have been analyzed using the older method of dynamic analysis using dynamic finite element analyses. These dams included Lexington (now Lenihan), Stevens Creek, Guadalupe, Calero, and Almaden. The analyses were completed in the late 1970s and early 1980s (Wahler, 1978, 1981, 1984). It should be noted that new concepts in earthquake faulting within the Santa Clara Valley have been developed since the original analyses were completed. Anderson Dam was analyzed using the simplified dynamic method of analysis in 1996 (R. L. Volpe & Associates). It is recommended that all District dams be analyzed using simplified dynamic methods by 2005. Then, once these analyses are complete, it is recommended that the dynamic stability of all dams be reviewed every 5-7 years, or more frequently if warranted, to assess new methods of analysis or other changes in criteria.

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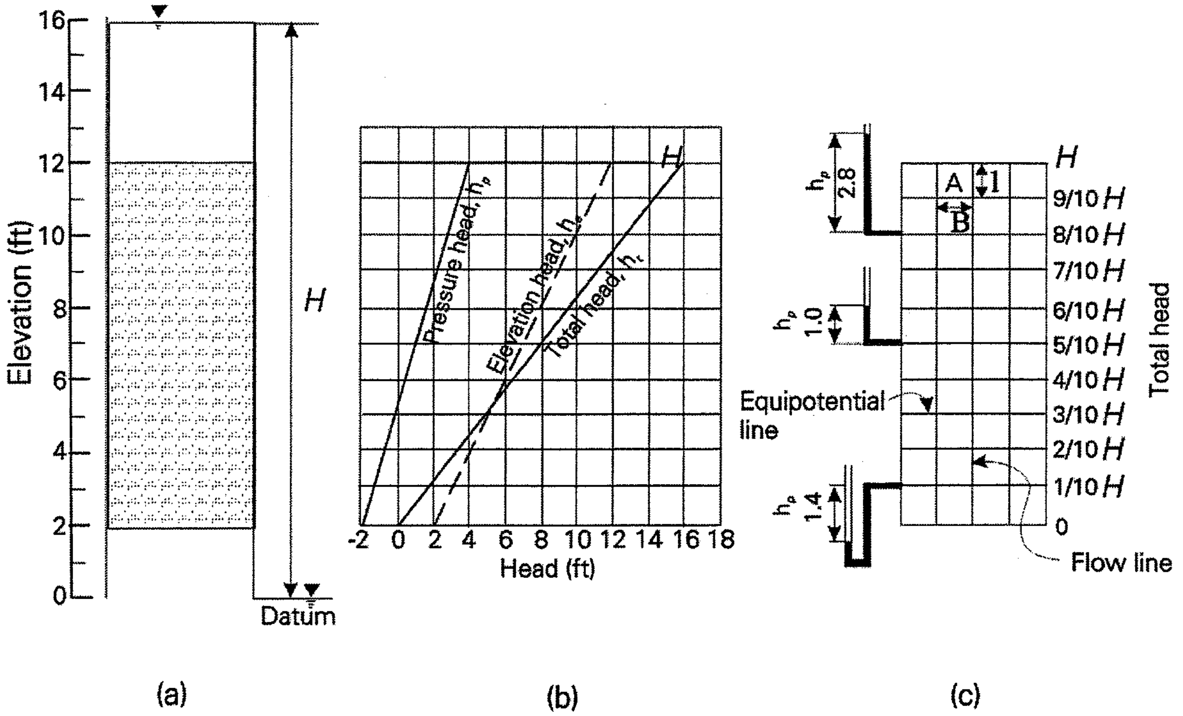

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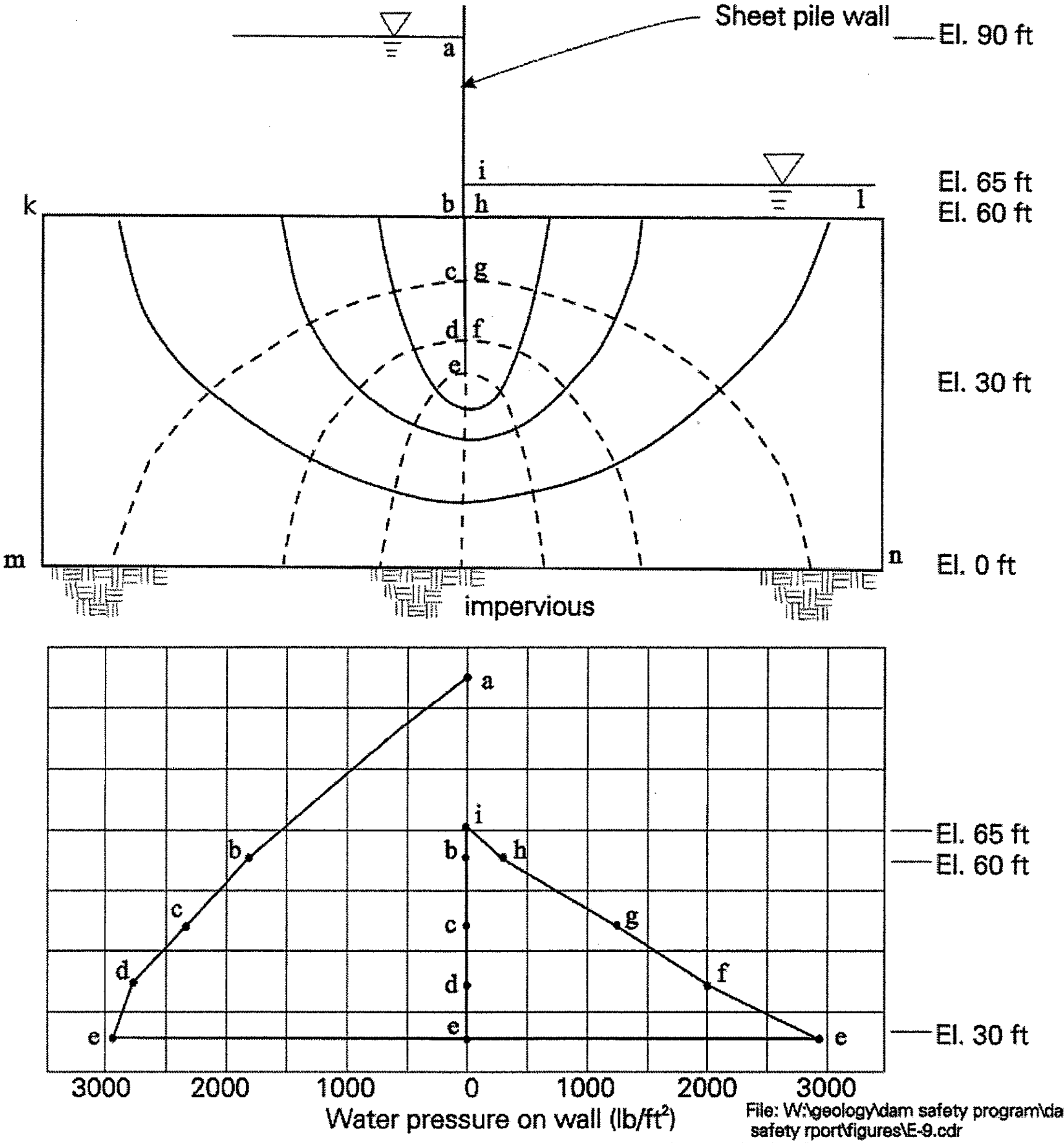
DATE 3/21/02
DESIGN RLV
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

Example of one-dimensional flow

SCALE None

FIGURE E-8



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DRAWN
JLN
CHECKED
RLV

Santa Clara Valley Water District

Example of Two-dimensional Flow
(Flow Under a Sheet Pile Wall)

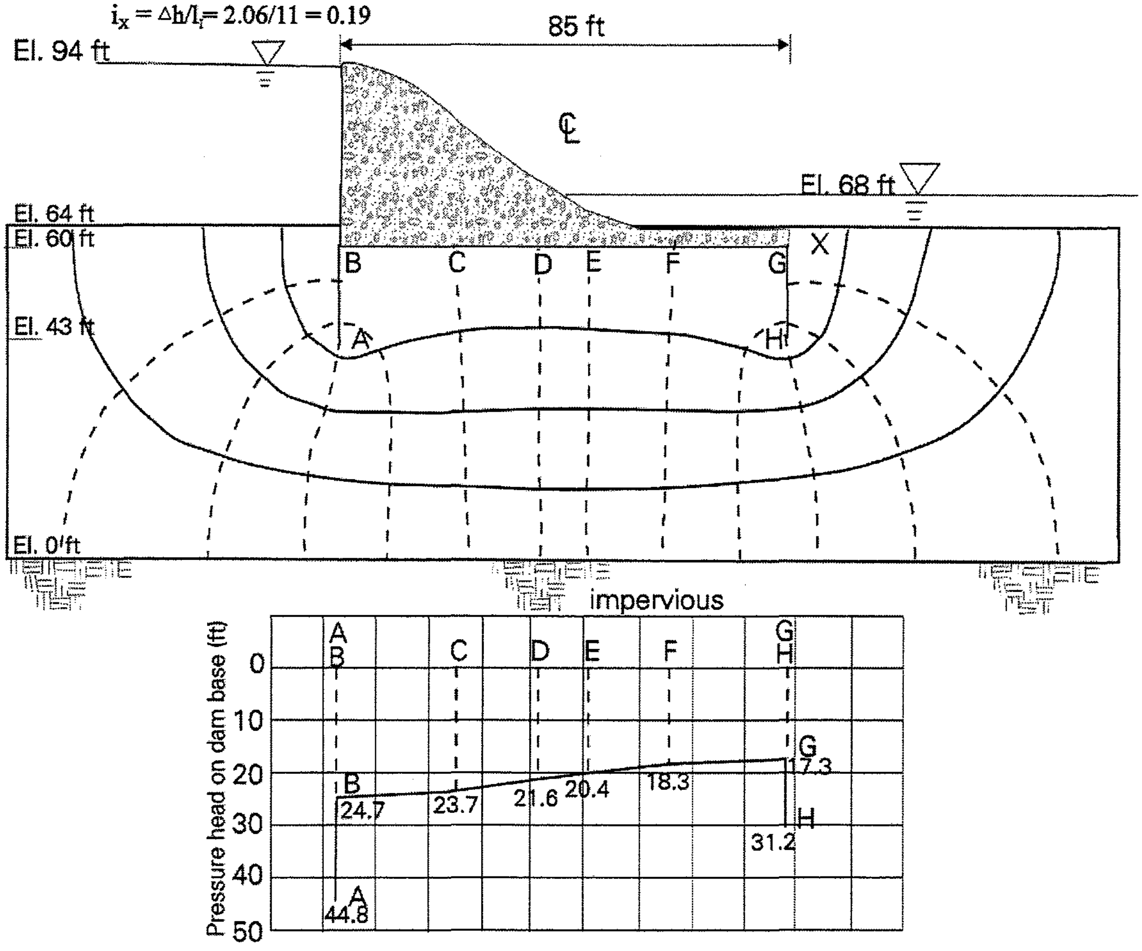
SCALE
None
FIGURE
E-9

nf = 4 nd = 12.6 k = 0.1 ft/min $\oint = nf/nd = 0.317$

$Q/L = kH\oint = 0.1$ ft/min x 26 ft x 0.317 = 0.825 $(ft^3/min)/ft$

Gradient in X:

$i_x = \Delta h/l_i = 2.06/11 = 0.19$



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3/21/02

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DRAWN
JLN

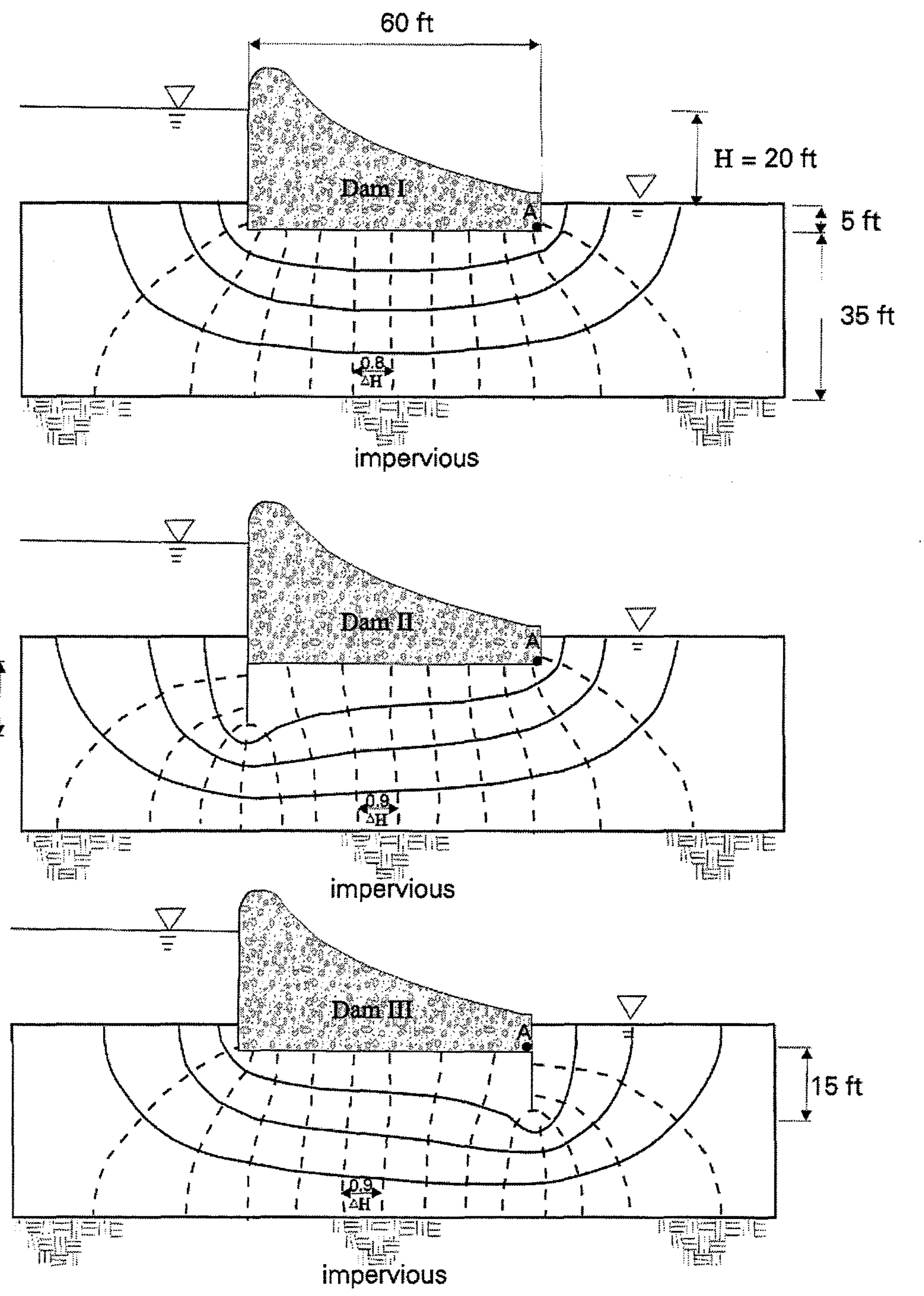
CHECKED
RLV

Santa Clara Valley Water District

Flow Under Concrete Dam

SCALE
None

FIGURE
E-10



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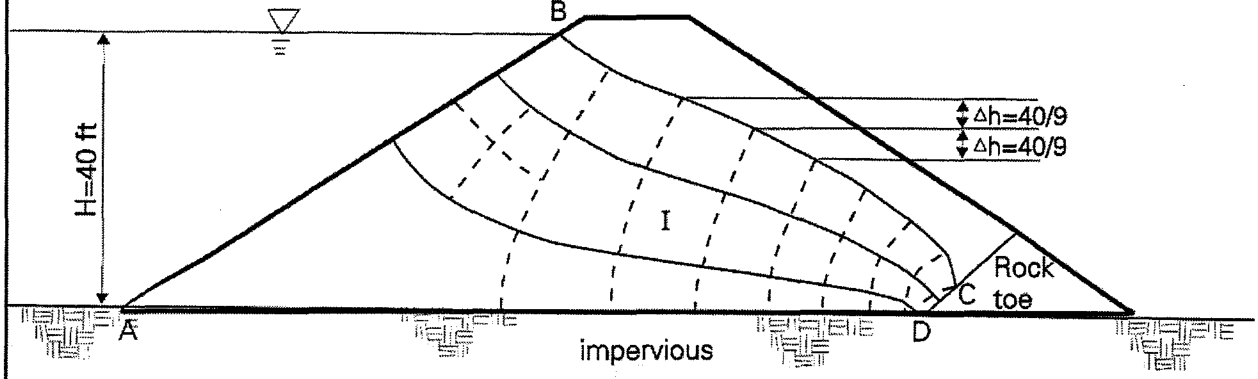
DATE 3/21/02	Santa Clara Valley Water District	Flow Under Three Concrete Dams	SCALE None
DESIGN RLV			FIGURE E-11
DRAWN JLN			
CHECKED RLV			

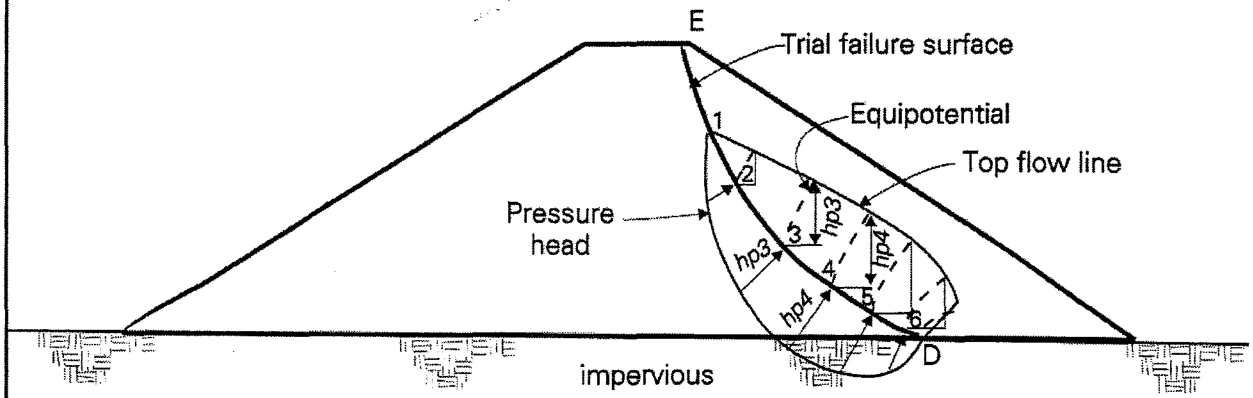
$$Q/L = kH\oint$$

where k = permeability = 0.0005 ft/sec, H= 40 ft, $\oint$= nf/nd =2.65/9 = 0.294

Therefore, $Q/L = 5 \times 10^{-4}$ ft/sec x 40 ft x 0.294 = 59×10^{-4} ft³/min/ft

The gradient in the square I equals $i_I = \Delta h/l_I = 40/9/11.2 = 0.4$





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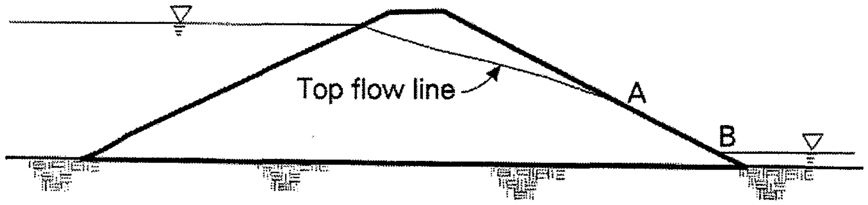
DATE 3/21/02
DESIGN RLV
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

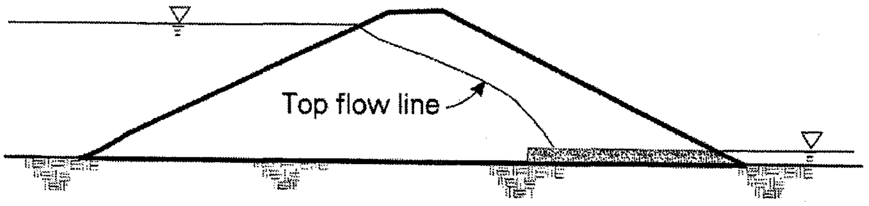
Flow Through an Earth Eam on an Impervious Foundation

SCALE None
FIGURE E-12

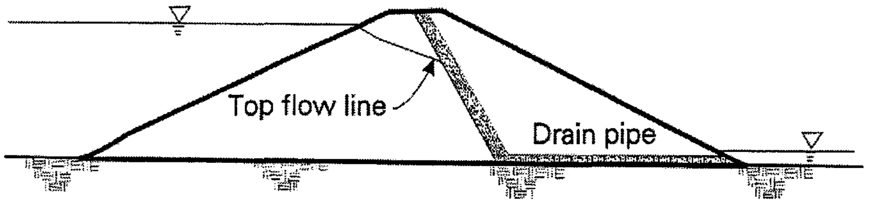
(a) Homogeneous dam without internal drain



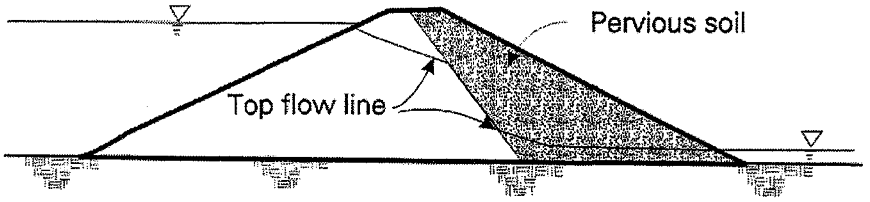
(b) Homogeneous dam with underdrain



(c) Homogeneous dam with chimney drain



(d) Zoned dam



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DESIGN
RLV
DRAWN
JLN
CHECKED
RLV

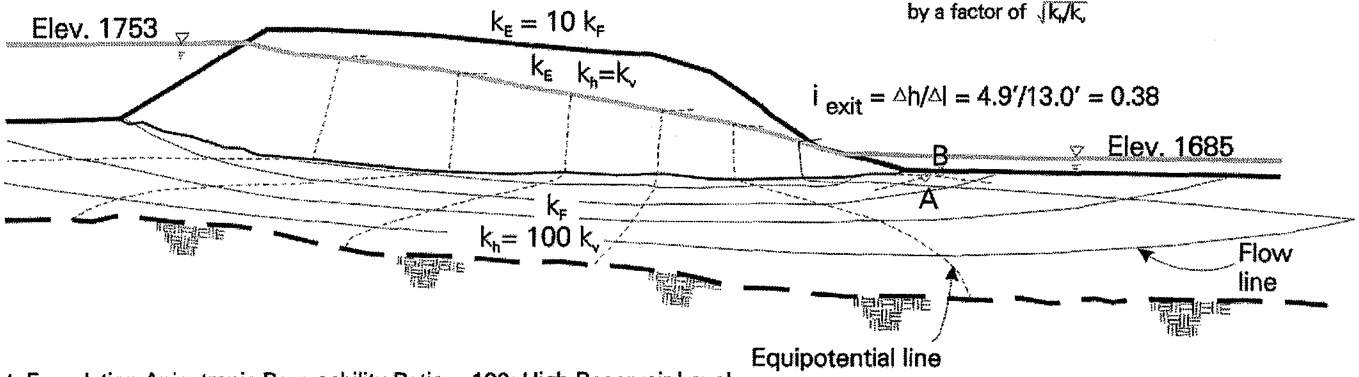
Santa Clara Valley Water District

Types of Internal Drainage
for Earth Dams

SCALE
None
FIGURE
E-13

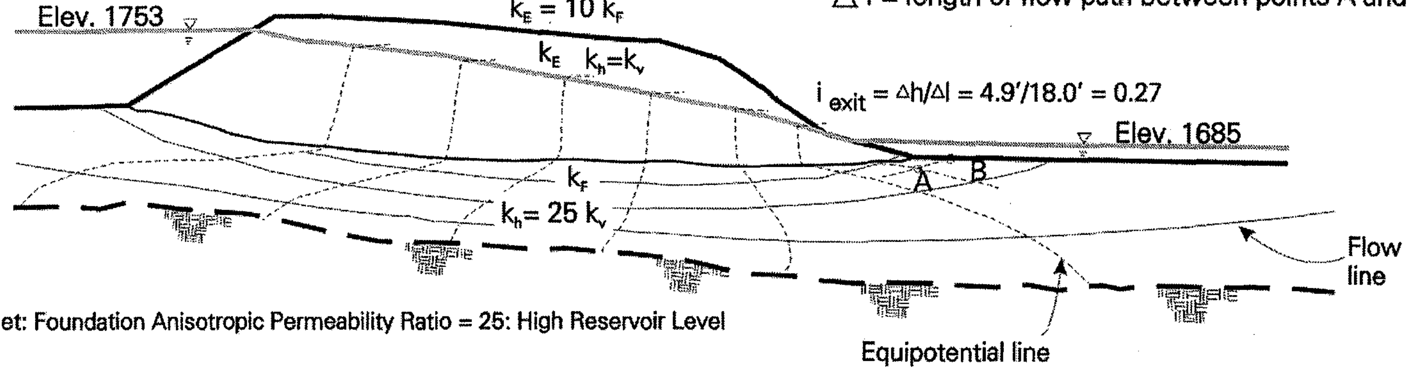
Notes: 1) equivalent embankment permeability
$= k_E = k_h = k_v$
equivalent foundation (sludge) permeability
$= k_F = \sqrt{k_v * k_h}$

2) these sections are drawn to true scale
the original flow nets were drawn on
transformed sections which were constructed
by expanding the depth of the foundation
by a factor of $\sqrt{k_h/k_v}$



Flow Net: Foundation Anisotropic Permeability Ratio = 100: High Reservoir Level

$\triangle$ h = head loss between points A and B
$\triangle$ l = length of flow path between points A and B



Flow Net: Foundation Anisotropic Permeability Ratio = 25: High Reservoir Level

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Santa Clara Valley Water District

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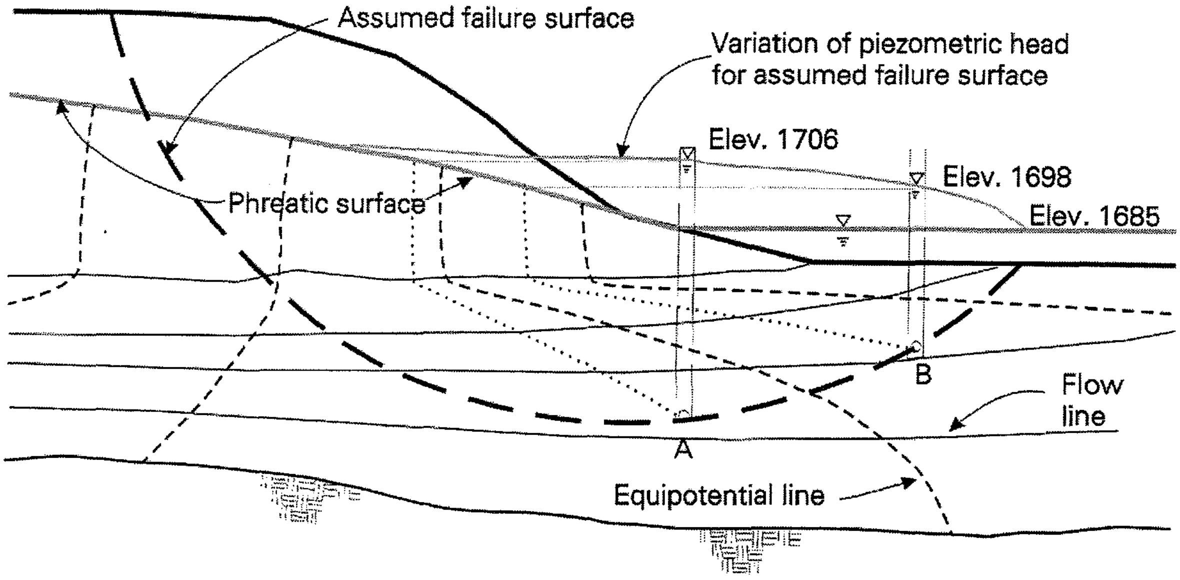
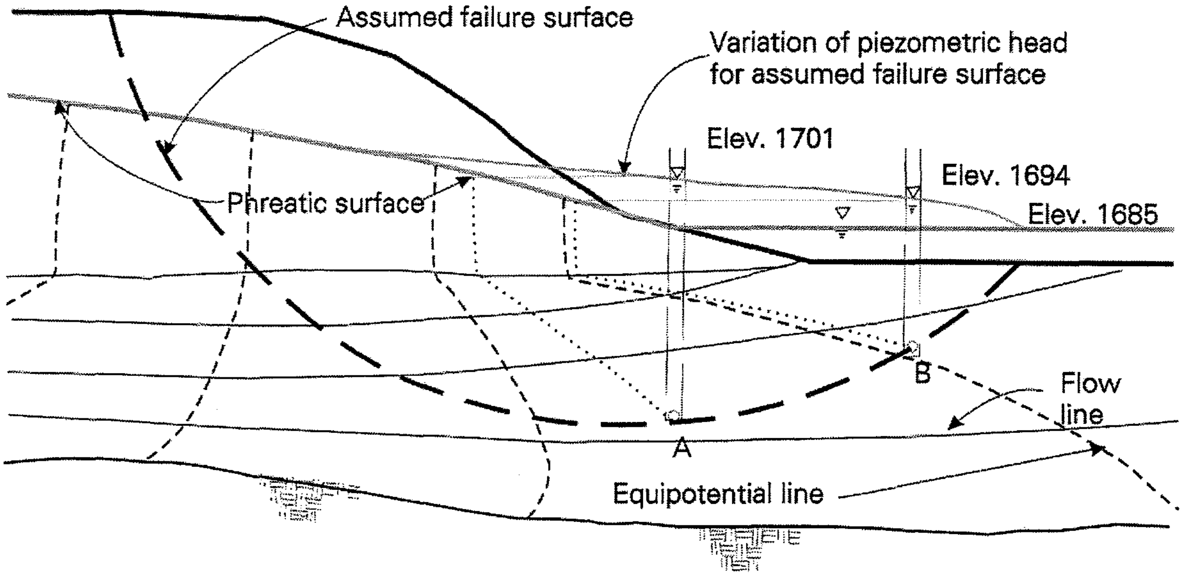
DESIGN RLV
DRAWN JLN
CHECKED RLV

Typical Flow Nets for Highly Anisotropic Conditions

SCALE None
FIGURE E-14



Anisotropic Permeability Ratio = 100



Anisotropic Permeability Ratio = 25

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DATE
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RLV

Santa Clara Valley Water District

Method of Determining Pore Pressure
from Flow Net

SCALE
None
FIGURE
E-15

APPENDIX F

AUTOMATED DATA ACQUISITION SYSTEMS

APPENDIX F
AUTOMATED DATA ACQUISITION SYSTEMS

This appendix presents a discussion of Automated Data Acquisition Systems. A majority of the text for this section is reprinted from a recently completed Guideline for Instrumentation and Measurements for Monitoring Dam Performance (ASCE, 2000). The text has been augmented as appropriate as it applies to District dams.

1.0 History and Background

For purposes of this report, an Automated Data Acquisition System (ADAS) is defined as any system for monitoring the performance of a dam that includes components for data collection that are permanently installed, and programmed to operate without human intervention. This definition specifically excludes the situation in which a portable recorder is periodically taken to the dam to collect instrument data. However, the definition is not limited to systems that include all potential aspects of ADAS: automated collection of data, transmission of the data to a remote site, and processing and plotting of data. For example, a simple system, including a data logger or computer located at the dam, and programmed to automatically record measurement data, would be included under this definition of ADAS, even if the data were subsequently retrieved and processed by personnel visiting the site and uploading the data manually.

ADAS equipment has been used for monitoring dam performance since the early 1980s. Some of these earliest installations of ADAS using industrial data loggers included Grand Coulee Dam (Bureau of Reclamation), Libby and Chief Joseph Dams (Seattle District of Corps of Engineers), and the original Lock & Dam 26 (St. Louis District of Corps of Engineers). It appears that the earliest automatic readout system of dam instrumentation was installed at Libby Dam in 1972. These systems recorded readings of extensometers to monitor potential dam movements.

One of the distinguishing limitations of these earliest systems used for dams was that they were generally constrained to multi-channel measurements of a single instrument type. In addition, equipment location was limited by a requirement for AC power and the inability to connect various data acquisition equipment into networks. This limited the ability to economically access the expanse of instrumentation and to bridge the disparate environments in which instruments are often deployed on dams. These earliest installations revealed the limitations of then-existing standard product data acquisition equipment developed for general industrial and scientific applications.

By the mid 1980s, more and more ADAS systems were being installed on several large dams, some of which by using specifications developed by dam owners or architect-engineer firms. These projects included the Bath County Pumped Storage Project (Virginia Power Company), Clarence Cannon Dam (St. Louis District Corps of Engineers), and Ridgeway, Fontenelle, Navajo, Morrow Point, Crystal, Glen Canyon, Flaming Gorge and Monticell Cams (Bureau of Reclamation). The ADAS specifications were developed carefully around specific project requirements. However, the list of potential suppliers was very limited, and even those with appropriate technology and products were unable to satisfy all of the specified requirements.

During this mid 1980s period, the Corps of Engineers took a lead in developing guidelines for the use of ADAS for monitoring the performance of dams. The Bureau of Reclamation also developed broad experience with multiple vendors for the installation and operation of ADAS systems. In addition, hydropower utility companies in the U.S. and Canada installed systems on several large dams. There were mixed results during this period, however, ranging from substantial success to outright failure of performance. Some of the significant difficulties included contractors who did not perform, equipment that performed poorly or required excessive maintenance, and ADAS manufacturers that either went out of business or vacated the dam monitoring business because of inadequate market potential or inappropriate technology. The large dam owners persisted, however, and by the late 1980s, successful installations became more common. Also, many of the earlier problem installations were corrected by installing new ADAS equipment using instruments and sensors that were better suited or more reliable for automated measurements.

Since the late 1980s there has been an increasing use of ADAS for dams owned and operated by commercial and private owners. With rapid advances in technology, ADAS equipment is continually becoming more reliable and more affordable. As a result, there is an observable trend for increased use of ADAS for dam performance monitoring.

Some of the advantages offered by ADAS installations for dams are as follows:

- Frequent collection of data, thereby capturing dam performance data under varying loading conditions.
- Collection of data when manual collection would be difficult or impossible, for example, when site access is difficult because of heavy rain or immediately following earthquakes.
- Reduction of reader-error in data collection and/or transcription errors in data entry.
- Increase in accuracy over older instrumentation of 10 to 100 times.
- Rapid processing and plotting of data, allowing easier technical analysis.
- Ability to set alarms or threshold values for each instrument.
- Ability to automatically contact project personnel in the event of alarms.

However, it must be remembered that an ADAS only collects data from point measurements, which cannot be relied upon to provide a complete picture of structural behavior. The instrumentation system can only collect the information that the instruments and the ADAS were designed to collect. For example, an ADAS would not identify the occurrence of seepage at a new location or the occurrence of settlement or horizontal movement at locations that were not instrumented. These types of changes can be observed only by visual inspections. In addition, instruments or ADAS components can fail, or cease to perform within specifications. Therefore any instrumentation program, whether ADAS is employed or not, is not a substitute for periodic visual safety inspection of a dam. Regular visual inspections are still an essential part of any dam safety program. The purpose of an ADAS is to provide more opportunities for the extent and timeliness of analysis, not to curtail opportunities for effective engineering judgment.

As noted above, ADAS installations are often capable of collecting more data than can be practically collected without automation. However, the collection of more data in and of itself does not provide value unless the data are regularly processed and evaluated. So, an important aspect of a successful ADAS is the allocation of resources to evaluate the collected data.

It is also important that the data collected from the ADAS are periodically verified through manual interrogation of the instruments. Hardware and software malfunctions can and do occur, so periodic manual verification must be included in any ADAS program to insure that judgments concerning performance of the dam are based on accurate information.

Finally, it must be noted that an ADAS system will require periodic maintenance, calibration and repair, and appropriate resources must be allocated for these functions. Sensors, computer hardware, and communication equipment will malfunction and require maintenance, repairs, and sometimes replacement. In addition, ADAS systems are subject to external influences that can require maintenance or repair. These external influences may include lightning strikes, vandalism, extreme weather conditions, fire, and water damage. It is very important that any ADAS system be designed with redundancy. Databases and archived data especially should reside on more than one computer. A new trend is to have available ADAS results available through a secure and password protected site on the Internet. This approach has great merit in that instrumentation data are available at a moments notice from anywhere the interested person has access to a portable computer.

2.0 Applications

ADAS installations have been used in three types of applications:

- Performance and safety monitoring
- Early warning systems
- Operations and controls

The three applications are described briefly below.

2.1 Performance and Safety Monitoring

Most ADAS installations to-date for dams have been for the purpose of collecting data for performance and safety monitoring. In such systems the ADAS replaces or supplements manual collection of performance data. In some cases, all instrumental monitoring data for the dam are collected by the ADAS. In other cases, selected data are collected by the ADAS and other data are collected manually. For example, only key seepage flows or piezometric levels are monitored by the ADAS, or only key crack meters or strain gauges.

ADAS systems have been used to monitor performance data for both dams that have been judged to be performing satisfactorily, and dams with identified dam safety concerns. In the first case, the ADAS is part of the routine monitoring program for the dam, while, in the second case, the ADAS may be a key part of the monitoring program used to determine whether further actions need to be taken because of the safety concern. In either case, the collected data are reviewed by qualified dam engineers to evaluate the performance of the dam relative to expected performance or dam safety performance standards (e.g. acceptable pore water pressures and acceptable seepage quantities).

2.2 Early Warning Systems

In some instances, ADAS equipment has been designed to include early warning features. The warning systems are intended to provide some protection for the public by initiating timely warnings to downstream populations that would be affected by certain occurrences at the dam (e.g., dam failure and large spillway discharges).

To facilitate early warning, the ADAS includes a computer-based data processing system that evaluates the collected data and issues warnings or alarms if certain threshold values are exceeded. For example, an alarm could be issued if the reservoir level rose above a critical elevation, seepage exceeded a preset limit, or piezometer readings exceeded specified values. Some system implementations allow combinations of alarm conditions involving multiple instruments to assure that a single-point failure or artifact event does not generate a false alarm.

The warnings are usually issued to operators or emergency management officials who evaluate the data and take action, as appropriate. In some unusual cases when sufficient time would not be available for operators and or officials to intervene, the warnings would be issued directly to the affected downstream population through public warning sirens or in-house radios, telephones, or alarms.

Tolt Dam, discussed later in this chapter, is an example of an ADAS that includes early warning features.

2.3 Operations and Controls

In a few cases, ADAS for dam instrumentation has been installed with links to SCADA (Supervisory Control and Data Acquisition) systems used for dam and reservoir operations. As a point of clarification, the Data Acquisition function of SCADA is to provide the critical data in real-time required for a supervisory Control strategy, involving an interactive human operator. SCADA is optimized to serve as a day-to-day operational tool in an attended control room environment. It is not the tool of the engineer who analyzes data, indeed different data, to make observations and judgments about the performance and safety of the hydraulic structure.

The tie-in of the dam ADAS with SCADA is important when some of the structural or environmental data becomes time critical data and must be available to the operator to meet safety or regulatory requirements. The human judgment involved with reservoir operations takes into account observable parameters such as reservoir elevation, inflow, freeboard, power generating capacity and demand, contractual issues, and both minimum and maximum environmental constraints on downstream releases. For certain types of operations, such as pumped storage plants, it is often desirable in the interest of safety to make some of the geotechnical and structural dam instrumentation data available to the operations people normally located in the power plant control room. To provide critical data of this nature, it may be desirable to link timely measurements from the dam monitoring ADAS to a reservoir and power operations SCADA system.

At the other end of the spectrum from operator-attended dams and reservoirs, there are many remote dams that are unattended, and fitted with control devices such as release gates and outlet works valves of various types. Many owners are beginning to equip these remote dams with

ADAS systems for dam safety. In some cases, they have added SCADA operational features to the dam safety aspects of the systems. In cases where the ADAS performs operational or control functions, the ADAS software includes logic routines to evaluate critical parameters. Based on these evaluations, the ADAS can either communicate information to human operators to make operation changes (e.g., open or close gates or valves), or directly issue signals to electronic actuators connected to the control devices.

3.0 Data Acquisition System Components

Modern electronic data acquisition systems are designed around system architectural concepts that have evolved slowly. These concepts, or models, for electronic systems are based on the state of current technology for electronic hardware, software, data communications, and current sensing and physical measurement methods. Major changes in technology often create the opportunity for manufacturers to develop products based on a new model, which serves certain applications in a better way. A new model may lead to the development of products that are more economical, easier to install, or perhaps more reliable in applications that have been difficult in the past. The new model, then, usually results in higher user acceptance.

Dam monitoring has historically proved to be one of those difficult applications. The revolution in technology over the past 10 to 15 years in computers, communications, and low-power electronics has enabled more cost effective, rugged, and reliable ADAS for dam monitoring. As a result, most new dams under construction today are designed with ADAS as an integral part of the geotechnical and structural instrumentation plan. In addition, many existing dams with active instrumentation programs for performance monitoring and dam safety are being retrofitted with ADAS. This section describes standard product ADAS equipment and measurement methods that are in use today (1999) as a result of the technology revolution of the past 15 years. It should be recognized that currently employed equipment and methods will continue to evolve and improve.

3.1 Distribution of Instrumentation of Dams

As noted in earlier chapters, geotechnical and structural instrumentation is typically installed at various points throughout a dam in order to observe the multiple aspects of structural response to loading conditions. The result is often a wide volumetric distribution of instrumentation: along the axis, in downstream and upstream offsets from the axis, and in elevation. Time-correlated measurements of these widely distributed instruments must be made periodically and brought together at a point where observations can be analyzed as a whole.

The distribution of dam instrumentation often presents a dilemma for ADAS. It is necessary to minimize the length of signal cables between the instruments and the measurement equipment for compound reasons. One reason is that long cable runs often contribute to degradation in the signal generated by the instrument, resulting in inaccurate measurements. Errors can come from external effects, such as differing ground potentials between the instrument and the measurement equipment, or from introducing long conductors causing detrimental modification of the equivalent measurement circuit itself. The degree of error depends largely on the type of sensing technique used in the instrument, and the capabilities of the measurement equipment.

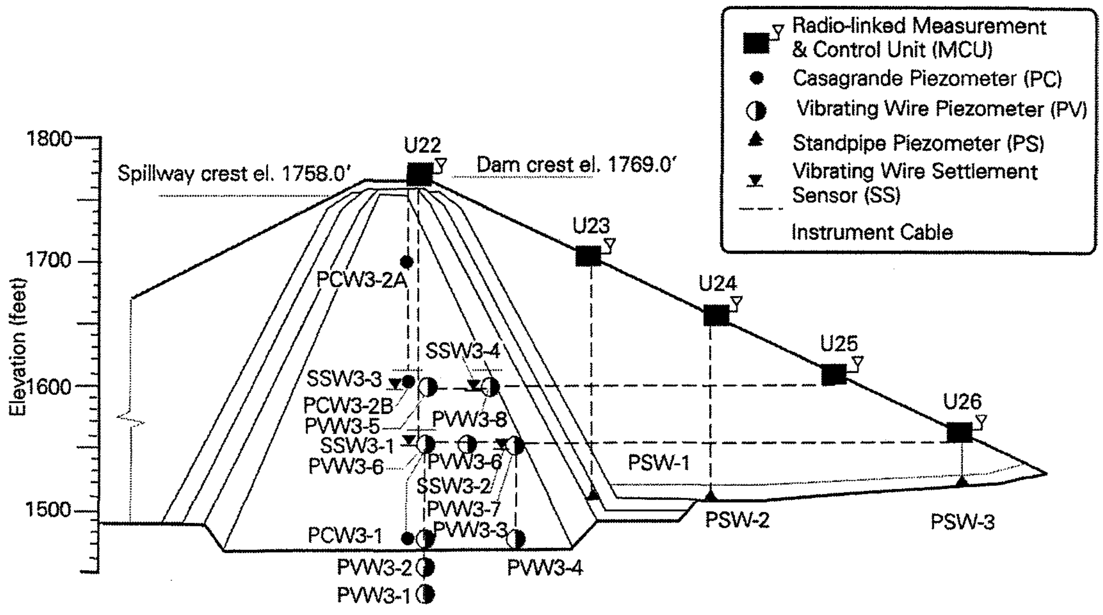

Figure F-1 - Instrument Connected to ADAS Components at the Diamond Lake Project (Formally Eastside Reservoir)

Another reason to minimize cable lengths is that long signal cables introduce high susceptibility to lightning damage for both the instruments and the measurement equipment. Furthermore, the cost of cable extensions and appropriate protection of the cables from physical damage can be very costly. All of the problems introduced by extended signal cables are far worse on embankment dams compared with most concrete dams. For these reasons, measurement equipment needs to be located fairly close to the instruments, particularly in the case of embankment dam installations.

The result of dispersed instrument distribution is that several (or in some cases many) measurement units must be located around or within the dam to minimize the length of cable connections to instruments. The distributed measurement units do not necessarily need to be complete data loggers in themselves. However, they need to perform the basic instrumentation, signal conditioning, and data conversion functions so that the data can then be conveyed to centralized data collection resources without data degradation. Planning and installation for the distributed equipment situation is fairly easy for most concrete dams that have galleries, adits, vertical shafts, and power distribution throughout. On the other had, it can be difficult for existing embankment dams that were not planned with ADAS prior to construction.

An installation configuration for an embankment dam where the ADAS system was designed in conjunction with the initial instrumentation plan for the dam is shown in Figure F-1. Note that the measurement equipment is located on the downstream embankment along the instrumented sections. Except for the Casagrande Piezometers and Standpipe Piezometers, which require vertical wells by design, cables are run laterally from the instruments to the measurement units. The measurement units require a good quality ground connection for the transient protection devices to be effective. Better grounding can usually be accomplished at lower elevations on the embankment. Installations on embankment dams done in this manner have proven to be far more reliable than those with long cable runs near the crest, running parallel to the axis of the dam.

3.2 General Ways of Acquiring Data

Readings from distributed measurement units must be collected in a common database so that all the instrument data, measured by equipment at different locations, can be analyzed together on the same time scale, and therefore under the same loading conditions.

The simplest of ADAS implementations is shown in Figure F-2. In this case, each measurement unit is a stand-alone data logger, requiring the data to be retrieved manually from each logger on a periodic basis. This retrieval is commonly done either by exchanging the data storage media with one that is blank, or else plugging in a potable computer and extracting the data from fixed storage in the data logger. Each data logger has a built-in calendar clock, and a date-time stamp (dts) is recorded along with each measurement record. It is important to assure that the clocks in each data logger are synchronized so that the comparative analyses of data are valid. (Equipment terms such as data logger, MCU, RTU, and DCP are clarified in Section 3.3.

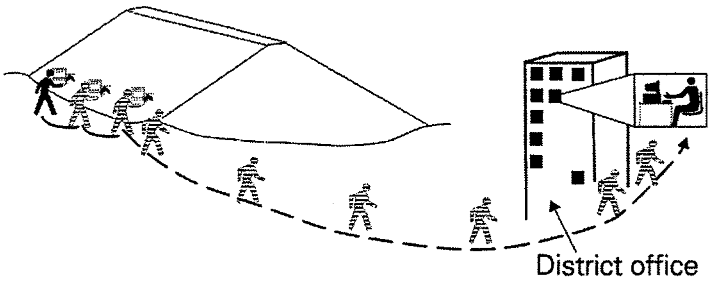

Figure F-2 - Normal Data Retrieval from Multiple Data Loggers (ASCE, 2000)

The situation in which multiple field measurement units are networked together at the dam by metallic cable, fiber-optic cable, or wireless links is illustrated in Figure F-3. This allows data from all instruments connected through multiple field measurement unites to be logged to a single, accessible location. Access to recorded data may by manual, such as in the prior situation, or the data may be accessed periodically using conventional voice or data communication facilities. An operator at a computer distant from the dam may manually initiate access over temporary connections such as the telephone network. Access may also be automatic, through polling software at the operator interface computer, or else originating from the field, which has the advantage that the field-installed equipment can initiate unscheduled connections in the event of alarm conditions.

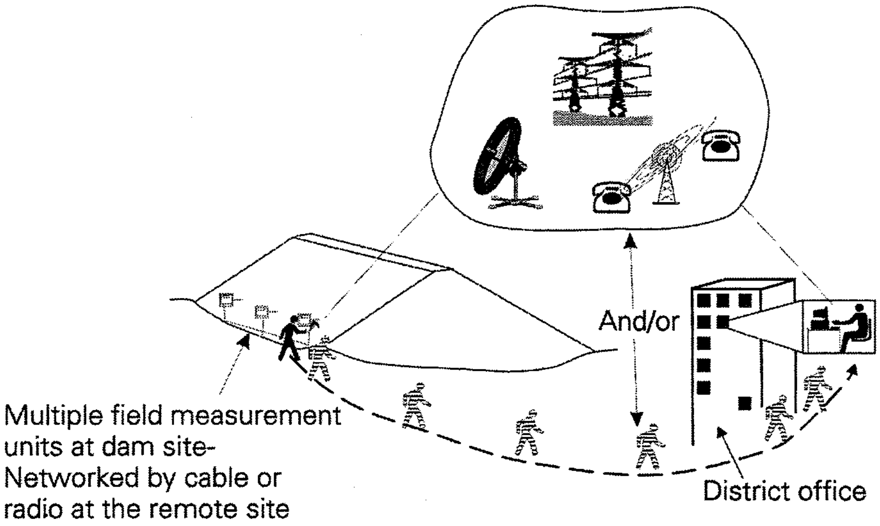
And/or
Multiple field measurement units at dam site- Networked by cable or radio at the remote site
District office

Figure F-3 - Manual or Telephone Data Transport from Field-Networked Measuring Units (ASCE, 2000)

An outline networked ADAS in which data transport to the operator interface computer is fully automated and essentially real-time is illustrated in Figure F-4. A routing or bridging node within the field-networked system must provide a connection to the wide area network that transports the data to the distant computer. This wide area network can be a satellite network, the Internet, an intranet or other privately owned and operated network, such as microwave. Hydropower utility companies often operate private networks.

The operator interface computer may be located at the dam, where there is an attended facility, such as a power plant control room or operations office. The more general situation, where there is an on-line computer at the dam site and another on-line computer at a distant office for data presentation and analyses, is illustrated in Figure F-4.

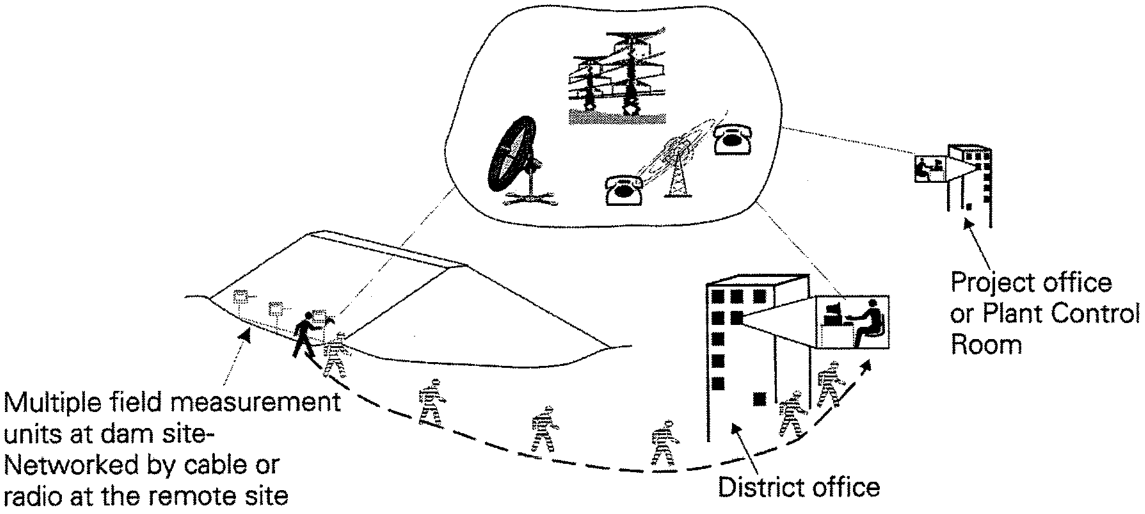
Project office or Plant Control Room
Multiple field measurement units at dam site- Networked by cable or radio at the remote site
District office

Figure F-4 - On-Line Data Transport from Field-Networked Measurement Units (ASCE, 2000)

3.3 Field Data Acquisition Equipment

The types of field measurement units historically used for automatic data acquisition on dams have included Data Loggers, Measurement & Control Units (MCUs), Remote Terminal Units (RTUs), and Data Collection Platforms (DCPs). These are not different manufacturers' names for the same class of instrumentation system component, but rather industry-standard (non-proprietary) terms for field units of a distinct class and, in most cases, application origin. Evolving operational features within each class of these units has resulted in a trend of migrating and overlapping capabilities. However, our discussions will stick to the fundamental origin of capabilities for these different classes of units to render a clearer understanding of the basic operating characteristics.

For some dam ADAS situations, Data loggers are the best solution. For others, MCU's may be required to achieve operational objectives. For still other, RTU's may meet the monitoring objectives in the most cost-effective manner. DCP's, on the other hand, are used on most dams owned and operated by federal agencies in the US for hydrological, rather than geotechnical and structural measurements. DCP's transmit data through a standardized environmental satellite communications facility.

General-purpose Data Loggers or MCUs may be able to perform the functions of the DCP, but they are usually not the most economical solution for hydrological data collection alone, where the dam operator has access to the GOES satellite data collection facility. The important point is that there is not a single class of field data acquisition equipment that performs all measurement and data transport functions in the required manner or with equal efficiency in all situations. The total instrumentation deployment, the environment, and the monitoring objectives determine the most appropriate class of field unit, or perhaps even mix of field unit types.

4.0 Unraveling the Terminology Tangle

The industry names that have been adopted for different classes of field data acquisition equipment were principally motivated by their operational role in the total distributed data acquisition system. Since the dam monitoring application is likely to invite proposals from companies with differing backgrounds, the civil engineering industry can expect to see proposals for products with different names for the same monitoring requirement. The following discussion introduces in a basic way the rationale behind the different names used for field data acquisition units. The field unit names make more sense when they are understood in the context of their behavior in the total distributed system. After understanding the operational characteristics of different field units, it is easier to recognize their implications for total system behavior and their effect on monitoring objectives.

4.1 Data Logger

In its basic form, a Data Logger is not a distributed system. In fact, the distinguishing feature of a Data Logger for our discussions is that it is a single data acquisition system in and of itself. A Data Logger is designed to be the end destination for the logging of data, even though the data are usually transferred to an external computer for display, reporting and ultimate storage. Most products called Data Loggers are not designed to be networked in the sense of being interconnected into a larger multi-node system with shared resources, such as a common data logging destination. Most Data Loggers do, however, support a terminal protocol for transferring stored data over direct digital of modem connections, without requiring the exchange of data storage media.

4.2 Remote Terminal Unit (RTU)

In contrast with Data Loggers, RTUs are not designed to log data, that is, to record data in an historical array with internally generated date-time stamps. RTUs have data storage registers that usually store only the most recently measured data from all connected instruments. The RTU register array is polled through a communication port by a host computer, which acquires the current data; other registers may be mapped to a command with parameters or data to execute remote supervisory control functions. The RTU is specifically not a logger, but rather a remote I/O terminal for the host computer. Therefore, the name Remote Terminal Unit is a precise representation of its function in hose-driven system architecture. Note that the logging resources for data acquisition functions reside in the host computer and not the field unit. Therefore, the host computer and the communication links are essential for the system o function as a data logger.

SCADA (Supervisory Control and Data Acquisition) is a term that universally implies a host-driven system architecture with slaved remote terminals. SCADA systems are also referred to as master-slave system-level architectures. As a side not, PLCs (Programmable Logic Controllers) have never proved appropriate for measuring dam instrumentation, but are often referred to as RTUs when they are employed as remote terminals for interfacing with discrete I/O (on/off status or control signals) in SCADA systems.

4.3 Measurement and Control Unit (MCU)

MCUs are similar to user-programmable data loggers in the way they process measurements, calculate results from measurements, and evaluate and execute control functions. In general, the processes executed by MCUs are time-bassed, although they may also be event-based. Unlike Data Loggers, MCUs are designed to be networked, and operated over networks to the full extent of their capabilities. Therefore, MCUs often provide more in the way of internal network services than data Loggers, often including repeating, bridging, and store-and-forward routing of both data and command messages. From a data acquisition perspective, the difference between MCUs and Data Loggers is that MCUs are generally not intended to be the final destination for automatically logged data. Rather, MCUs provide temporary local data-history storage for buffering between data transmissions, with the routine logging destination being a computer on the network. This computer runs software for maintaining centralized data stores and other system operations.

The term MCU was adopted by the industry to differentiate this class of instrument from an RTU. RTUs normally operate as slave I/O terminals, with the measurement and control scheduling, and data logging provided by a software program running on the host computer. On the other hand, MCUs posses the programmability and intelligence to execute measurement and control functions autonomously, independent of communications with a host computer.

From a communications standpoint, MCUs generally operate according to a node-driven system architecture in which each node makes its own determination as to when it can access a shared-media network. RTUs, on the other hand, do not speak until they are spoken to be the host.

4.4 Data Collection Platform (DCP)

DCPs are designed to measure environmental data (primarily hydrological and meteorological data), and transmit it by UHF radio from remote locations to a satellite-borne radio transponder. Data are relayed to a ground receiving station by a geosynchronous weather observation satellite. DCPs are designed for periodic transmission of limited data sets in a standardized format through pre-assigned time windows. Unlike other field data acquisition units described in this section, DCPs are designed around a specific satellite communications facility, and transmit data only (DCP capabilities for receiving limited incoming messages are rarely used). These facilities are currently government owned and operated in the United States and around the world. In the U.S, it is called the GOES (Geostationary Operational Environmental Satellite) system. In the United States and elsewhere, government agency sponsorship is required to use these facilities. Use and operation in the U.S. is coordinated by NESDIS (National Environmental Satellite, Data, and Information Service), part of the U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA).

DCPs provide a uniform mechanism to collect reservoir level and other hydrological data for reservoir operations and flood warning, independent of whether or not the dam has geotechnical and structural instrumentation. This reliable system is in widespread use around the world.

4.5 Remote Monitoring Unit (RMU)

RMU is not an industry-standard term for a class of data acquisition equipment, in that it currently lacks the elements of an industry standard regarding voluntary (non-proprietary) use among multiple manufacturers. However, the term warrants discussion for reason that it has been used in specifications and publications pertaining to ADAS for dam instrumentation.

Beginning in the mid-1980s, specifications were written by civil engineers for several dam monitoring projects in the U.S. At the time, it was not clear to the engineers how manufacturers would respond to the Requests for Proposals (RFPs) for these projects. Since there was not widespread experience in the civil engineering community with ADAS for dams, it was also not clear what class of data acquisition equipment was most suitable. To avoid biasing specifications in favor of a particular architecture, and at that time perhaps a specific manufacturer, it was decided by the drafters of these specifications to introduce a generic term in RFP specifications for ADAS on dams. RMU is a term, then, which may be appropriate to use in an RFP, when the engineer would prefer not to take full responsibility for the design of the system in a way that specifies the system architecture. Using the term RMU is a way to solicit proposals for designs from prospective system suppliers.

5.0 System Configurations for Dam Performance Monitoring

In the following subsections, three system-level models are described that represent the most common current practice for ADAS on dams. It is important to recognize three assumptions that underlie the choices of these models.

- We are addressing data acquisition that includes multiple points of geotechnical instrumentation - not just one or two points of hydrological data are being addressed.

- The models include instrumentation that may be deployed over an extended hydraulic structure. For this reason, the models address the general case of a distributed system of multiple field measurement units, since these units should not normally be located too far from the instrumentation that is connected to them.
- The models address a system of automated data acquisition which includes not just the measurement and recording of data, but also the mechanism for transporting that data to a computer where it can be viewed, reported to various personnel, and analyzed.

These three assumptions are based on the nature of the general dam monitoring situation. The first assumption of a distributed system for all models addresses the general case. However, any of the three models can be collapsed to consider the case of a small instrumentation program that simply requires one field unit at a single location for instrument connections. The operational features of each model remain valid for the simple case.

5.1 Data Logger System Model

The first model considers the simple data logger. A data logger is defined as a field unit used to measure multiple electrical signals (voltage, resistance, current, etc.) over extended periods of time. Measured data are converted into a digital format and stored, along with the date and time of measurement, on a local or internal mass storage device. The key aspect of a data logger is that the normal storage mechanism is local to the field unit. The data must then be conveyed to a computer database, generally by one of two general methods illustrated in Figures F-5 or F-6.

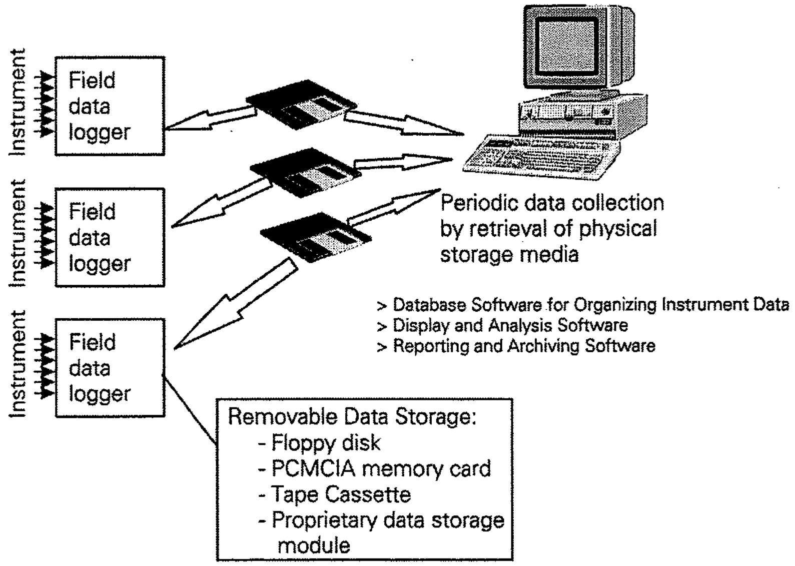
Instrument
Field data logger
Instrument
Field data logger
Instrument
Field data logger
Periodic data collection by retrieval of physical storage media
> Database Software for Organizing Instrument Data
> Display and Analysis Software
> Reporting and Archiving Software
Removable Data Storage:
- Floppy disk
- PCMCIA memory card
- Tape Cassette
- Proprietary data storage module

Figure F-5 - Data Logger System Model with Manual Data Transfer (ASCE, 2000)

The simplest method of data transfer is shown in Figure F-5, in which data are retrieved from the data logger on some type of removable storage media, commonly a floppy disk, an industry-standard memory card, digital cassette type, or the data logger manufacturer's proprietary data storage module. The recorded media is retrieved from the data logger, and then taken to a computer to load the data into a database. The database is accessible to one or more software programs for display, analysis, reporting and archiving.

The term “SneakerNet”, although classified as jargon, is at least a concise way of referring to a network of one or more runners who collect and bring home the data from remote data loggers.

SneakerNet has some clear advantages compared with strategies requiring communication connections for automatic transport of data to computers. There is a capital cost advantage since the additional equipment and/or communications services associated with automatic data transport are not required. Another potential advantage is that there is no restriction on the location of the data logger imposed by the requirement for accessibility to a communication link. The person collecting the data can presumably also perform some visual inspections while visiting the logger locations.

There are also some obvious disadvantages to SneakerNet. First, the requirement for routine physical accessibility may require installing the data logger in a compromised location in order to make short connections to instruments. Secondly, timeliness of data observations may be compromised by inaccessibility to remote sites. Changes or new trends in measurements will not be recognized until the data are transferred to the computer and analyzed by graphical or limit screening methods. Another disadvantage of protracted delays between human observation is that equipment or instrument malfunctions may not be detected for extended periods of time, which may conflict with the monitoring objectives. Finally, there is the ongoing personnel cost associated with manual data retrieval.

The disadvantage of SneakerNet can be overcome by accessing the data logger over the telephone network, as shown in Figure F-6. Data loggers are typically equipped with a serial communications channel for connection to a serial COM port on a personal computer. This allows programming the data logger with configuration software usually provided by the manufacturer, and extracting data from the logger memory and/or data storage device. Along with the serial communication channel, data loggers usually provide a communications protocol for modem connection control over telephone networks, and a terminal protocol for transferring data from the on-board data storage to the computer. This allows the same configuration and data transfer functions between the computer and the data logger over the telephone network that can be done with a local serial cable connection. Variations in the logger-embedded modem control software may also allow connections using cellular telephones or communication satellite terminals.

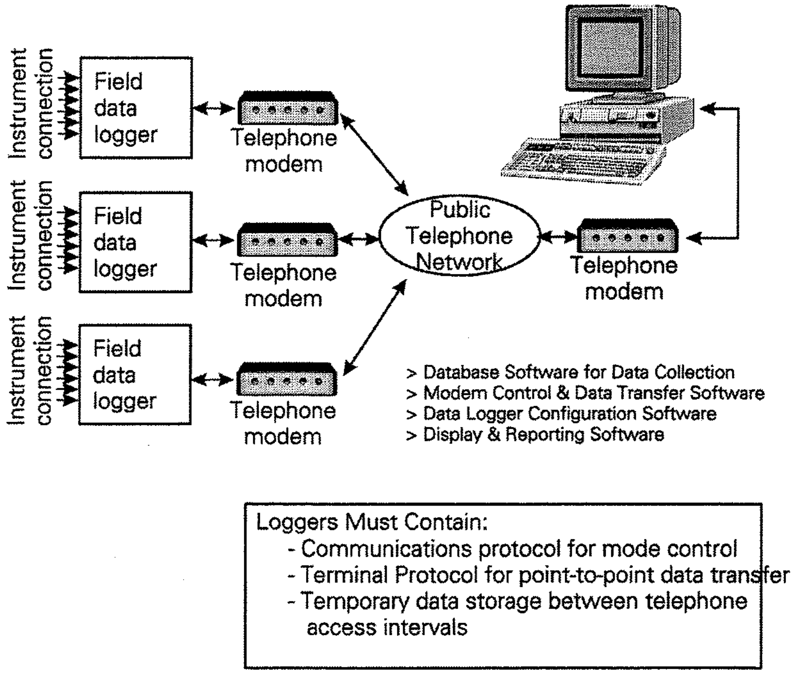
Instrument connection
Field data logger
Telephone modem
Instrument connection
Field data logger
Telephone modem
Instrument connection
Field data logger
Telephone modem
Public Telephone Network
Telephone modem
> Database Software for Data Collection
> Modem Control & Data Transfer Software
> Data Logger Configuration Software
> Display & Reporting Software
Loggers Must Contain:
- Communications protocol for mode control
- Terminal Protocol for point-to-point data transfer
- Temporary data storage between telephone access intervals

Figure F-6 - Data Logger System Model with Telephone Data Transfer (ASCE, 2000)

The Public Switched Telephone Network (PSTN) can provide a convenient and economical method for retrieving data from remotely located data loggers on a far more frequent basis than SneakerNet would allow. Data Loggers normally operate in a terminal mode, with the computer operating in host mode. With this type of data retrieval, an operator at the computer usually originates and terminates the telephone connection. Therefore, this is also considered a manual method of data transfer, since transferring the data requires operator initiative.

The primary drawback of this configuration comes when there are multiple data loggers required on a project, which is often the case on a large dam instrumentation program. This model assumes a separate telephone line connected to each data logger, increasing the system cost and complexity.

5.2 Host-Driven Network Model

In host-driven networks, the host computer controls the time and frequency of measurements and the flow of data from RTUs to the host, as shown in Fig F- F- Because of its centralized role as arbitrator for all system communications and I/O operations, the host is often referred to as a central computer. The sequence of interrogating data from RTUs is referred to as polling the RTUs. This network architecture is referred to as master-slave. Where the host computer is the master, and the RTUs are slaves. SCADA systems, for example, almost always employ host-driven networks, since the primary purpose of a SCADA system is to provide data to a human operator in the process of operator-supervised command and control of remote I/O. In other words, SCADA is designed primarily to be operator interactive with fast response time over remote communication links, and these objectives are usually best served with centralized system architecture.

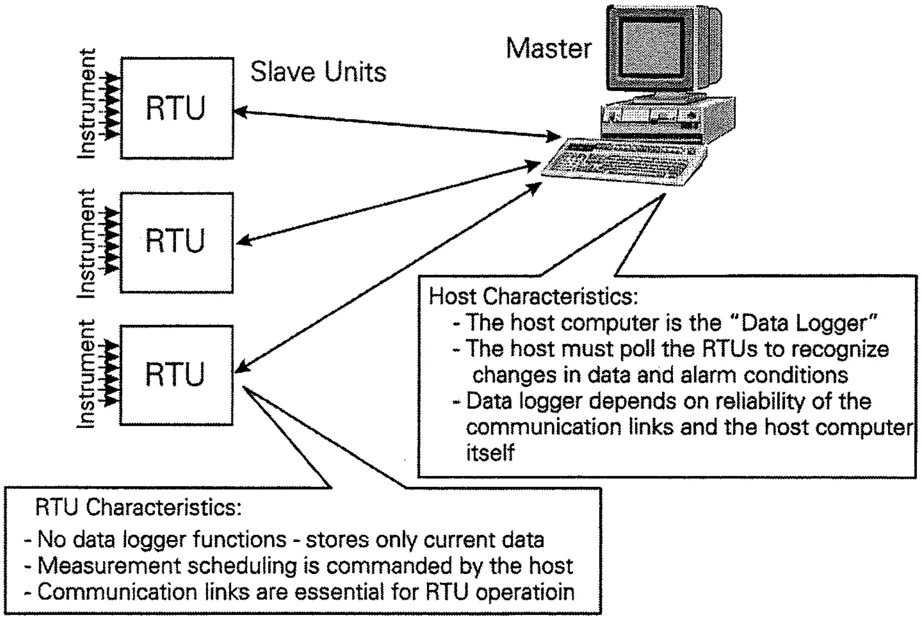
Master
Slave Units
RTU
RTU
RTU
Instrument
Instrument
Instrument
Host Characteristics:
- The host computer is the "Data Logger"
- The host must poll the RTUs to recognize changes in data and alarm conditions
- Data logger depends on reliability of the communication links and the host computer itself
RTU Characteristics:
- No data logger functions - stores only current data
- Measurement scheduling is commanded by the host
- Communication links are essential for RTU operatioin

Figure F-7 - Host-Driven Networks - Distributing Critical Functions (ASCE, 2000)

Host-driven networks are simpler, at least in concept, than node-driven networks described in the next section. It should be recognized that a host-driven network distributes essential functions across the network which are required for logging data at the user-prescribed intervals. So if a communication link fails, or an unattended central computer fails or becomes unstable (for example, must be rebooted, as often happens with Windows PCs), data records will be missed during these periods of equipment outage. Note that the host computer itself is the data logger in a host-driven ADAS. All data logging functions, logic, and data storage reside in the host computer.

Host-driven networks have the disadvantage that substantial investment is usually required in the central computer and the communication links to achieve across-network reliability. This level of investment is not generally appropriate for unattended ADAS without a supervisory control imperative.

Questions often arise regarding the suitability of SCADA systems for ADAS on dams, because SCADA is the most common system architecture employed for industrial instrumentation networks. However, while host-driven networks are conceptually simple, they can be costly to implement with adequate reliability in ADAS situations for dam instrumentation.

It should be noted that a local field-bus network of intelligent instruments, polled by a data logger or MCU, is in fact a host-driven network. However, this type of local instrumentation network, often installed as one or more sub-networks, is not typically burdened by the cost disadvantages of remote SCADA host-driven networks described above. Typical field-bus networks often benefit from industry-standard communication protocols, are quite reliable, and do not suffer from the cost impacts cited above. Unlike PCs, the data logger or MCU host controller is (or should be) designed to be a hardened unit, that is, to run unattended in remote areas with high reliability. Furthermore, the communication links between the master controller and slave intelligent instruments are local area networks, secured within the confines of the project, with high reliability, persistent connections. By contrast, dial-up telephone links are not persistent.

5.3 Node-Driven Network Model

Node-driven networks are commonly referred to as peer-to-peer networks. Figure F-8 illustrates the configuration of a node-driven network. As implied by the peer-to-peer terminology, nodes share communications capabilities equally without a master controller, and each node can initiate access to a shared communications medium. To cite a familiar example, the Ethernet standard (IEEE 802.3) is a peer-to-peer network, and provides node-initiated Media Access Control (MAC) for computer Local Area Networks (LANs).

Node-driven networks are employed in instrumentation systems to allow critical functions at the node to operate autonomously. In data acquisition systems, intelligent nodes can often perform their automatic data acquisition functions more reliably without requiring communications with a distant host computer. In normal operations, the remote nodes initiate the communication with the operator interface computer rather than the other way around. This way, the measurement node retains the data in a temporary queue until a reliable communication link is established with the remote computer. In addition, measurement nodes can report to the computer, not only on a periodic basis, but also upon detecting an alarm condition. Furthermore, node-driven remote equipment consumes far less power since the generally power-consumptive communication resources do not need to be continually active, as they would be to respond to polling sequences from a host computer.

To summarize, the beneficial characteristics of MCUs operating as peers in node-driven networks are:

- The remote units will perform data acquisition even if the communication link with the operator interface computer is unavailable.
- The power consumption of the remote equipment can be managed effectively when nodes control access to communication links, which are often the highest power consuming components of the remote equipment.
- Nodes can be designed to bridge between different communication media, such as wire line, radio, telephone and satellite, increasing flexibility and reducing overall system deployment cost.

6.0 Instrument Interfacing

The typical geotechnical and structural instrumentation used for dam performance monitoring introduces special interfacing requirements for ADAS equipment. Dam instruments have not traditionally included integral signal conditioning and other electronic circuitry to generate electronic output signals that are compatible with the inputs of general industrial and scientific ADAS equipment. Avoiding embedded electronics in dam instrument designs continues to be the practice of the industry, because many dam instruments cannot be retrieved and re-calibrated after installation. Therefore, there is a prevailing requirement for simplicity in geotechnical and structural instruments, minimizing total parts count, and avoiding as much as possible parts that can fail or have a tendency to long-term drift.

The result is that ADAS used for dam monitoring must include signal conditioning for the mix of instruments that are to be monitored. The flexibility and efficiency of how the signal conditioning is implemented can be a significant factor in the cost, reliability, and accuracy of the ADAS. Ten to 15 years ago, it was typical for manufacturers of geotechnical instruments to offer data loggers which would handle the signal conditioning only for instruments of their own manufacture. Since the late 1980s, however, multiple manufacturers of universal ADAS systems have developed products or product extensions specifically for dams that can handle the unique signal worldwide. It is this universal instrument connectivity which has contributed more than any other factor, to the success of ADAS for dam performance monitoring.

7.0 Communication Interfacing

Data communication links are a major consideration for most monitoring situations that employ more than a simple data logger. Communication interfacing is complicated by the fact that the optimum equipment distribution, considering cost, reliability, and other factors of performance, often requires multiple types of data communication links or media. As an example, the network connections for the embankment dam section shown in Figure F- 1 are illustrated in Figure F- 8. One of the important features of the node-driven network is that alarm conditions can be detected locally at MCUs without requiring polling from centralized host computers. Remotely located measurement units in node-driven networks can usually be designed to consume substantially less power than their host-driven counterparts. Another benefit is that most online node-driven networks employ source-node data buffering. Distributing the eggs in multiple baskets has the effect of providing operational redundancy for protection against data loss in the event of a communication link or a computer failure, or the unavailability of a non-persistent communication link.

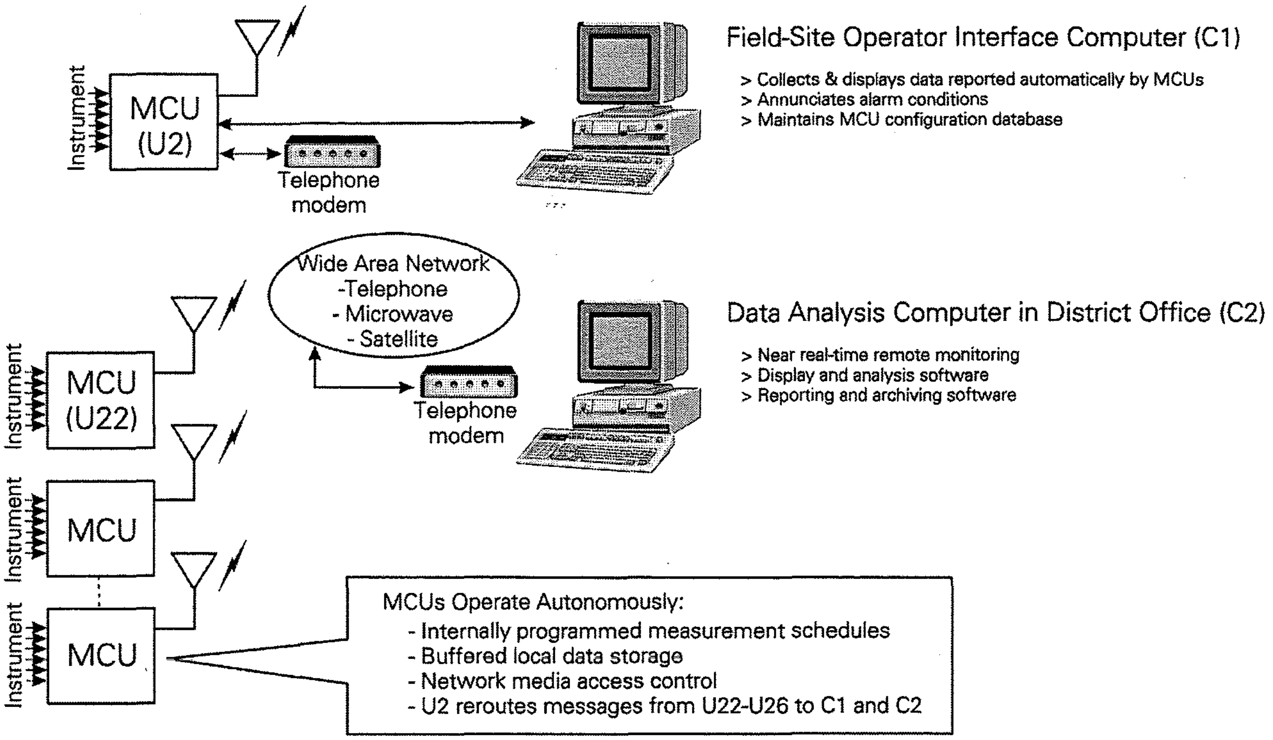
Instrument
MCU (U2)
Telephone modem
Field-Site Operator Interface Computer (C1)
> Collects & displays data reported automatically by MCUs
> Annunciates alarm conditions
> Maintains MCU configuration database
Wide Area Network
-Telephone
- Microwave
- Satellite
Telephone modem
Data Analysis Computer in District Office (C2)
> Near real-time remote monitoring
> Display and analysis software
> Reporting and archiving software
Instrument
MCU (U22)
Instrument
MCU
Instrument
MCU
MCUs Operate Autonomously:
- Internally programmed measurement schedules
- Buffered local data storage
- Network media access control
- U2 reroutes messages from U22-U26 to C1 and C2

Figure F-8 - Node Driven Networks (ASCE, 2000)

8.0 Design Considerations

An ADAS system should be carefully designed before installation. It is often said that instrumentation should no be installed just for the sake of instrumentation, but rather the instrumentation system should be designed for specific purposes. Each instrument should be intended to answer specific performance questions and instrument readings should provide operators with information they need to make decisions regarding the operation of the structure. Similarly, an ADAS should not be installed simply to automate, but rather should be carefully conceived to fulfill a specific purpose or purposes.

The following is a series of questions that should be considered to properly design and ADAS:

- What is the purpose of the ADAS?
- How long is the ADAS system required to function? – For a short period of time to evaluate a specific dam safety or performance concern, or for the life of the structure?
- What are the conditions at the site and how will they affect the ADAS? Weather, public access, etc.
- How many instruments will be included in the ADAS?
- What kind of instruments will be included?
- Is power available at measurement locations? If not, how will power be supplied?
- What communication methods are required for an economical and reliable installation within and around the dam? What communication facilities are available between the site and observer locations?
- Who will operate the system, and what is their level of sophistication with respect to electronic equipment and computers?
- How will the system be maintained?
- How frequently will data be collected?
- Will data be transmitted to remote locations and if so, how often?
- How frequently will data be processed?
- What type of reporting is required?
- What hardware and software components are most appropriate for the dam?

9.0 Practical Considerations

Modern ADAS equipment for dam monitoring consists of standard products which are site-configured using appropriate options, and programming the equipment to meet the data acquisition objectives. However, there are always variations in field conditions from site-to-site that warrant special attention in system planning, design and installation. Experience has shown that there are three issues with remotely installed ADAS equipment that must be properly considered on a site-specific basis to avoid reliability and maintenance difficulties. These issues requiring special attention are power, environmental protection and transient susceptibility.

9.1 Power Sources

Most remote data acquisition equipment used for dam monitoring (whether Data Loggers, RTUs) is battery operated and designed for minimal power consumption. This allows the possibility of installing the equipment near instrument locations where there is often no availability of house power. The equipment is operated from the battery, and an on-line battery charger is used to maintain the battery in a fully charged condition. The normal batter operation has the added benefit of allowing the equipment to continue operation (non-interruptible power) even in the event of intermittent interruptions of the battery charging power source.

The battery chargers for remote ADAS equipment are typically designed to use either AC power or solar power for inputs to the charger system.

Obviously, if reliable AC power is conveniently available at the equipment location, this source should be used for on-line charging. However, in many locations on and around dams, AC power is not conveniently available, and battery charging is typically accomplished with a solar panel because of its low cost and good reliability.

Reliability problems with remote power sources typically creep into the installation when the battery and/or the solar panel are sized inadequately or marginally for the installation. Since power consumption of the remote equipment can be highly variable, depending on installed options, communication links, and programming, a degradation of power resources can occur over time if equipment configurations or programming are changed. When changes do occur from the original plan, more is usually demanded of the equipment rather than less, by way of additional instruments requiring additional power-consuming signal conditioning, and increased duty-cycle of the radio transmitter.

Engineers involved with ADAS designs must be conservative when sizing batteries and solar panels for remote installations. Attention should be given to the worst case anticipated load, the Equivalent Sunshine Days (ESD) for the installation site, and the lowest ambient temperature that the battery will endure. The battery technology and charging characteristics, particularly as they relate to temperature, must also be considered. Applications engineering data and assistance should be available from the equipment manufacturers and systems installation contractors and integrators.

9.2 Environmental Protection

Electronic equipment will not operate properly and will be seriously damaged from water or moisture intrusion due to condensation. Therefore, particular attention must be given to outdoor installations of ADAS equipment. Bringing multiple instrument cables into a weatherproof enclosure that also provides easy access to the equipment can present difficulties if the field conditions are not fully anticipated when the equipment is purchased.

The variations in quantity and sizes of instrument cables, appropriate space and mounting considerations for transient protection and communication options, and climatic conditions must be considered in selecting appropriate field enclosures. Resistance to vandalism must also be considered for many locations. The variations in these requirements from site-to-site can present some challenges for equipment manufacturers whose objective is to offer standard weatherproof enclosures that are suitable for most situations. There is usually less expense and less risk involved in purchasing properly engineered enclosures from the ADAS equipment manufacturer. However, this is true only if the manufacturer has a standard offering that meets all the site-specific requirements. This is not always the case, and custom solutions are sometimes required.

There are basically two approaches to environmental protection for outdoor installations of electronic equipment. One approach is to try to keep the dry air in and the moist air out with a (somewhat) sealed enclosure. The other approach is to design the enclosure to naturally aspirate, or breathe, to eliminate a significant temperature differential between the inside air and the outside air. Natural aspiration (convection) is assumed because motor aspiration, such as used in traffic signal control boxes, is not practical for solar powered sites.

10.0 ADAS System Considered for Dam Safety Program

This chapter has presented a discussion on the current state-of-practice for ADAS. This section summarizes the District's experience with the ADAS installed at Lenihan Dam and how this experience has shaped our considerations for the remaining ten dams that are a part of the Dam Safety Program. It is clear with the pioneering work at Lenihan Dam that the District has come a long way in developing a working ADAS to significantly enhance its overall control of dam safety at one District dam. It is equally clear that much work remains to bring all of the District dams to a similar level of control in the area of dam safety.

10.1 Existing ADAS for Lenihan Dam

Lenihan Dam has afforded the District a unique opportunity to interact with a "state-of-the-art" ADAS system, at least with respect to earth dam engineering. The Lenihan Dam system has been in continuous operation since March, 1999. With few exceptions it has performed very well, and the information it has provided has proved invaluable for our dam safety assessments. There are currently 54 piezometers, 13 in-place inclinometers, a reservoir level gage and one strong motion accelerometer (sma) connected to the Lenihan Dam ADAS. The instruments are read every six hours, and then a daily average is computed and saved to the database. The sma is used as a triggering switch such that in the event of a moderate or stronger earthquake, the ADAS is programmed to increase the sample rate to once every fifteen minutes in the event of an "alarm" condition. Alarm conditions are triggered as a result of piezometeric levels that exceed defined thresholds, or by a peak ground acceleration measured at the crest exceeding 0.05g. The Lenihan system is a Node-driven network, with one node installed at the outlet, and two nodes installed in the Control Building at the crest. The three nodes are networked, with one node in the control building, called the "Gateway", facilitating communication to the outside world. The Gateway at Lenihan initiates a phone call once every day to download the previous days readings to a PC in the District offices. It will also initiate a download in the event of an alarm. Data is retrieved and stored in a relational database that can be used to view and analyze the instrument data.

Lenihan Dam has proved that automated data acquisition provides significant benefits over manual data collection. Manual data collection requires a lot of man-hours be expended to collect, validate and append a single reading onto those previously taken to establish normal "baseline" behaviors. In fact, this process is so time-consuming that sometimes it is never completed. In some cases our records of piezometric head or weir flow are nothing more than a pile of field sheets, so determining present dam behavior with historical behavior becomes a research project. The Lenihan Dam ADAS gathers the data, validates it, processes it collects it into a relational database. We spend our time viewing and analyzing information rather than collecting and organizing it. Furthermore, this information can be easily shared or customized for use by different people. We would like to see all of our instrumentation data from all of our dams compiled in an easily accessible database that can be used by WUE staff to assess dam safety. One way to accomplish this is to automate the data collection, validation and storage process utilizing ADAS systems at our other dams.

10.2 Future ADAS for District

When we first installed the Lenihan Dam system we knew very little about ADAS, and were dependent on outside consultants for guidance. Our experience with the Lenihan Dam system has provided us with practical knowledge that we can apply toward the design, installation and maintenance of future systems. For instance, over the three years of operation of the Lenihan Dam system, there have been numerous false alarms caused by faulty instrument readings. We would look for future systems to incorporate some failsafe procedure to determine when a reading is faulty to avoid these kinds of alarms. The Lenihan Dam system has also required a significant amount of maintenance since its installation. We recognize that all systems do require maintenance, but our selection of future systems will focus on making low maintenance a higher priority, even at the expense of a few features.

Most importantly, future systems should be capable of collecting, validating and storing data into a central database. The key to a well-designed ADAS is the way in which data are transferred into the database. The nature and design of this database is the most important factor in a successful monitoring program – since it is the information stored in that database we are ultimately left with. A properly designed database not only provides easily accessible information about the physical state of a dam, but also can perform system status checks and provide early indication of potential problems.

The collection and control of the ADAS database will significantly impact the Dam Safety unit within the WUE Unit. For example, it is conceivable that automating the instrumentation and data processing at ten more dams will require the establishment of an operations and control center for dam safety. This center would house the computers and other equipment necessary for communicating with the individual ADAS located at each dam. These computers would also be networked so that the data from each could be transferred to a central database. The current status of the instruments at each dam would be displayed on the individual computer monitors. This information would also be available to anyone with access to the central database. Secure Internet access to this database would also allow dam safety personnel to view the information from virtually anywhere. This would greatly enhance our alarm response capabilities by allowing dam safety personnel to respond to an alarm condition from wherever they happen to be when it occurs.

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APPENDIX G

PRELIMINARY INSTRUMENTATION DESIGN

APPENDIX G
DAM INSTRUMENTATION CONCEPTS

1.0 Introduction

This chapter presents a discussion of the geotechnical instruments most commonly used for monitoring dam performance. Techniques for instrument installation, operation, and maintenance are presented. Its goal is to present a solid overview of the current state of the art in monitoring instrumentation, but not to describe all aspects of every individual instrument.

The flow of information from the physical world to the instrument user is shown in Figure G-1. Instruments may be mechanical, electromechanical or electronic. In mechanical instruments the measured parameter is converted to a visual reading by purely mechanical means. Some mechanical measuring methods are extremely simple, such as the use of a bucket and stopwatch to measure flow or seepage at a dam. Electromechanical devices convert mechanical movement into a measurable electronic signal. An example is an extensometer in which movement of a rod anchored in a borehole operates an electronic transducer. Such instruments are currently being used at the District's Penitencia Water Treatment Plant to measure the magnitude of movement of the compression gages on the three large diameter pipes located at the toe of the landslide. In purely electronic instruments the measured parameter generates an electrical signal without an intervening mechanical step. The vibrating gauge piezometer is an example of an electronic instrument.

Most new instruments today are electronic or electromechanical. The electronic portion of these instruments combines a sensing element (sensor and transducer) and an electronic circuit (signal conditioning electronics) as illustrated in Figure G-1. The transducer converts the sensed parameter into an electrical signal. The electronics excite the transducer and transform its typically weak signal into an amplified output that can be recorded at the end of a long cable. The signal conditioning electronics are typically packaged as close as possible to the transducer in the same housing.

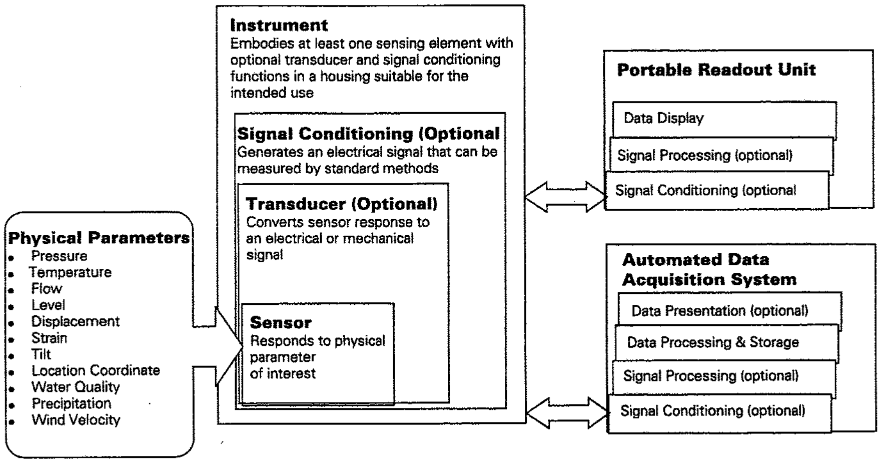
Instrument
Embodies at least one sensing element with optional transducer and signal conditioning functions in a housing suitable for the intended use
Signal Conditioning (Optional
Generates an electrical signal that can be measured by standard methods
Transducer (Optional)
Converts sensor response to an electrical or mechanical signal
Sensor
Responds to physical parameter of interest
Physical Parameters
Pressure
Temperature
Flow
Level
Displacement
Strain
Tilt
Location Coordinate
Water Quality
Precipitation
Wind Velocity
Portable Readout Unit
Data Display
Signal Processing (optional)
Signal Conditioning (optional
Automated Data Acquisition System
Data Presentation (optional)
Data Processing & Storage
Signal Processing (optional)
Signal Conditioning (optional)

FIGURE 4.1 Block diagram of generic instrumentation systems

However, some transducers, such as vibrating wire or fiber optic types, produce a signal that is relatively immune to electrical noise even when transmitted over cable lengths of many hundreds of feet. When such transducers are used, the signal conditioning electronics may be located near or within the recording or display unit.

Instrument readings may be recorded manually (by writing down or photographing the value displayed on a mechanical scale or digital readout), mechanically (typically by pen-and-ink lines on a circular or strip chart recorder) or electronically by a data logger or computer as illustrated in Figure G-1. The trend today is away from mechanical recorders and increasingly toward automated electronic measurements. The benefits of automated recording include a continuous data record, a variety of data processing options to improve accuracy, reallocation of labor resources to the more valuable functions of analysis and decision making, and the ability to automatically initiate alarms and other actions if critical thresholds are exceeded. A list of instruments that have been successfully used in dams is summarized on Table G-1.

2.0 The Origins of Modern Dam Instrumentation

In the United States, Roy Carlson designed an apparatus that would measure pressure in concrete and earth dams by means of a sensor consisting of a stretched wire. He measured the variations in electrical resistance of the wire relative to the movement of two anchoring points, using a Wheatstone bridge. This unbonded resistance-wire strain gauge was first produced commercially in 1932 and was later supplemented by piezometers, concrete stress cells, earth pressure cells and jointmeters.

An important milestone in the field was marked in 1931 in France when Andre Coyne, founder of the French consulting engineering firm Coyne et Bellier and designer of more than 55 arch dams, obtained a patent for the vibrating wire sensor, then called an acoustic indicator. Following this, 17 vibrating wire extensometers were installed in the Bromme arch dam (1930-1932), built over the Truyere River. The first major monitoring program was carried out on the Mareges arch dam (1932-1935), in which 78 extensometers were placed in the body of the arch and 40 more in the abutments (Coyne, 1938). At 90 meters in height, Mareges was, at the time, the highest double curvature arch dam in the world. Since then, this type of sensor, which has given rise to a whole range of instruments, as proven to be very well suited to the particularly severe environmental conditions typical in civil engineering. This sensor has developed a reputation for stability, accuracy and a long life span. Its frequency output signal makes it less sensitive to cable faults than other sensors (Bordes, 1998).

Beginning in the 1930's and 1940's, the use of vibrating-wire sensors for dam monitoring became a worldwide phenomenon. Maihak Company was founded in Germany in 1936. Andre Coyne founded Telemac in France in 1947. Telemac was followed in the early 60's by Geonor, in Norway. Carlson Gauges are still used extensively in Japan where Kyowa and Toyoko Elmes manufacture them; they were also used extensively in China, the USA and in South America but were gradually superseded in these countries as the advantages of vibrating-wire technology became more widely appreciated. This trend was accelerated by the founding in the U.S.A. of Irad Gauge in 1974 and Geokon in 1979. Both of these companies have made important contributions to modernizing and expanding the range and variety of vibrating-wire sensors used on dams throughout the world, in parallel with the improvements adopted by their European counterparts, Irad Gauge was later acquired by Roctest, founded in Canada in 1967, who also acquired Telemac in 1991. Carlson Gauges, made originally by Carlson instruments, are still manufactured by R. S. Technical Instruments in Canada.

Table G-1 Dam Monitoring Instruments and Parameters Measured

Instrument Name	Parameter Measured	Measurement Technique	Sensor/Transducer Type
Convergence Meter	Change of distance between two points at rock or concrete interface	Frame-mounted dial gauge and tape between points	Mechanical dial gauge or digital micrometer
Crack Meter	Opening or closing of crack in concrete	Monitoring displacement across crack	Grid-lines, micrometer, dial gauge, linear potentiometer, LVDT, vibrating-wire, inductive sensor with frequency output
Inclinometer (manual)	Inclination of borehole casing used to derive lateral movement of borehole	Inclinometer probe measures casing inclination at discrete intervals	Gravity-referenced servo-accelerometer; magnetostrictive sensor
In-Place Inclinometer	Inclination of borehole casing used to derive lateral movement of borehole	Simultaneous measurement of casing at multiple depths using multiple sensors	Gravity-referenced accelerometer, electrolytic tilt sensor, or vibrating-wire; magnetostrictive sensor
Joint Meter	Opening or closing of joint in concrete	Transducer embedded across concrete joint to measure displacement	Linear potentiometer, LVDT, vibrating-wire, inductive sensor with frequency output
Load Cell	Force perpendicular to cell	Insertion of cell into soil or between structural member	Hydraulic, resistance strain gauge bridge, vibrating-wire
Piezometer	Porewater pressure, uplift pressure	Level or pressure measurement in open standpipe, pressure measurement by sensor embedded in soil	Water level indicator (tape), pneumatic, hydraulic, vibrating-wire or resistance strain gauge on diaphragm, Bourdon gauge
Settlement Gauge	Change in elevation with respect to known point	Pressure measurement at bottom of fluid column or water level measurement in tube or chamber	Vibrating-wire sensor, strain gauge type pressure sensor, or optical reading of liquid level

Table G-1 Dam Monitoring Instruments and Parameters Measured (Continued)

Instrument Name	Parameter Measured	Measurement Technique	Sensor/Transducer Type
Soil Extensometer	Horizontal extension or contraction of fill or soil	Rod or wire anchored at one end, buried in trench	Micrometer, dial gauge, linear potentiometer, LVDT, vibrating-wire
Weirs and Flumes	Seepage	Water level (stage) measurement behind barrier	Ultrasonic level sensor, head gauge, submerged pressure sensor, float gauge, bubbler tube
Tiltmeter	Rotational movement	Sensor attached to concrete or steel member, or embedded in dam	Force balance accelerometer, electrolytic tilt sensor, vibrating-wire, magnetostrictive
Total Pressure Cell	Total pressure in soil	Steel diaphragm embedded in soil or on face of concrete structure	Strain gauge bridge, vibrating-wire, pneumatic, hydraulic

A large number of dams in the past have been instrumented using hydraulic and pneumatic sensors. These kinds of sensors continue to be used where automation is not required. Soil instruments and Geotechnical Instruments in England, plus Slope Indicator in the U. S. A. and Gloetzl in Germany, have been leaders in the development and manufacture of pneumatic and hydraulic instruments. In the 1980's and 1990's new types of electronic gauges, such as the electrolytic tiltmeters developed by Applied Geomechanics and installed in the District's Lenihan Dam, were also introduced to the dam monitoring field. The District installed its first vibrating gauge piezometer and electrolytic tiltmeter in Lenihan Dam in 1998.

Monitoring equipment used for dam instrumentation has to satisfy two challenging requirements. Initially, it must have a life span comparable to the design life of the dam. Secondly, it must be extremely stable with virtually no zero drift. The market or demand for dam monitoring equipment on a world scale is only modest at best. Consequently, there are only a few companies who can supply equipment able to meet these requirements. It appears, however, that this trend may be changing for the better

3.0 Future Trends

Dam monitoring encompasses the fields of sensors, electronics, computers, and information technology, all of which are advancing rapidly. Many of these advances will affect the way that dam monitoring is done in the future. A few predictions of future trends in each of these fields are given below. There will certainly also be developments that cannot be foreseen at this time.

3.1 Sensors and Electronics

Silicon-based sensors are becoming more and more widespread, driven by high-volume markets such as the automobile industry. In these sensors, known by the acronym MEMS (micro-machined electromechanical sensors), the transducer is photo-etched into a silicon chip along with much of the signal conditioning circuitry. Accelerometers, tiltmeters and pressure sensors have already been produced using this method. Magnetic and chemical sensors on a single chip are also on the market. To date, instruments based on this technology have not shown the precision or long term stability required for dam monitoring applications. However, continuing improvements may change this. The advantage of sensors of this type is that they are mass produced and therefore relatively inexpensive.

Whereas fiber optical data transmission is now widespread in the telecommunications industry, cost-effective fiber optical sensors have been slower to develop. A great advantage of fiber optical sensors is their immunity from electrical noise and transients. With continuing advances in the field, sensors of this type are likely to be used more for dam monitoring work in the future.

The technique known as time domain reflectometry (TDR) has been used increasingly for monitoring embankment and slope stability in recent years, but dam monitoring applications have been few. Originally developed to locate breaks in communication and power lines, data collection consists of simply attaching an electronic cable tester to a coaxial cable grouted into a borehole, then taking a reading. The basic principle is similar to that of radar. An electrical pulse is sent down the coaxial cable. When the pulse encounters a break or necking, which causes an electrical impedance change, it is reflected. The reflection shows as a "spike" on a plot of reflected signal vs. time. Because the pulse velocity in the cable is precisely known, the depth to the zone of deformation is easily computed. At present, TDR cannot easily identify the direction or magnitude of deformation, only its location along the cable. Advantages of TDR are simplicity and rapid data collection. It takes only a few minutes to read any TDR cable regardless of its length. Small diameter boreholes and inexpensive coaxial cable can be used. No casing is required; in fact, casing can mask the onset of any movement. These simplified features may lead to increased use of TDR for monitoring the condition of embankment dams and reservoir slopes.

Today, most electronic instruments used on dams put out an analog signal: a voltage, current or frequency that is proportional to the measured pressure, tilt, displacement or other parameter. This output is transmitted in analog form and is then digitized and recorded at a data logger or computer. Increasingly, however, instruments are entering the market with electronics that digitize the signal at the transducer and produce an output in a standard digital protocol, such as RS232, RS485, RS422 or SDI12. The advantages of digital vs. analog output are 1) direct interfacing to a personal computer, 2) more options for signal processing in the instrument to improve signal quality, 3) increased noise immunity during signal transmission over cables or radio and 4) transmission of the data in engineering units. Increased use of dam monitoring instruments with digital output is likely in the future.

3.2 Computers and Data Acquisition

Rapid increases in the power of personal computers, and the steady fall in their prices, will continue to propel the trend toward automated collection of dam monitoring data. The ready availability of powerful and affordable computers, coupled with the development of digital instruments, may reduce the reliance on specialized data acquisition systems in the future and lead to increased use of networked PCs for data acquisition. The proliferation of low-cost data loggers, some of which have networking capabilities themselves, could contribute to this trend. The use of devoted computers to receive and edit downloaded instrumentation data has been successfully used at the District since mid-1999. It has been determined that it is important to have these as stand-alone systems since they are connected by modem to the instrumentation computers at the dam. In this manner they are not impacted by system overhead or protocol difficulties.

Data transmission from instrument to acquisition system is now performed mainly by copper wires and, to a lesser extent, by radio telemetry. The use of cables on and around dams presents problems because of distance, rough terrain and other logistical factors, and susceptibility to lightning strikes. Use of radios overcomes these problems, but has been limited by cost and reliability issues. The development of new radio telemetry techniques, such as spread spectrum transmission, and lower-cost radios will lead to increased use of radio telemetry in the future.

At present there is little standardization in the way digital signals are communicated by radio. The increased use of radio data transmission in many industries will result in development of standards in data format and protocols, leading to lower-priced telemetry systems and interchangeable hardware for telemetry applications. Use of digital cellular and satellite technology is starting to become a widespread means of data transport also. Appendix F presents a detailed discussion on Automated Data Acquisition Systems.

4.0 Performance Specifications

When selecting a monitoring instrument, the user must first evaluate its published performance specifications in order to judge its suitability for the intended use. When interpreting measured data, performance specifications must be taken into consideration to differentiate meaningful readings from meaningless noise. Understanding performance specifications is therefore critical for getting optimal results from a monitoring program. Among the most important are range, resolution, accuracy, precision and repeatability. As these are sometimes confused, their definitions and significance are discussed below and illustrated in Figure 4.2. Performance specification should be traceable to known standards, such as those maintained by the National Institute of Standards and Technology (NIST) in the United States.

Range specifies the highest and lowest values that the instrument is designed to measure. For example, the range limit for a piezometer might be 0-50 psi. The expected range of the parameter to be measured must fall within the range limits of the instrument. A related specification, **Span** is the arithmetic difference between the upper and lower range limits. For example a range of ±50mm is equivalent to a span of 100 mm.

Resolution is the smallest change in the measured parameter that can be distinguished by an instrument. The resolution if typically many times finer than the instrument's precision, repeatability and accuracy and is never expressed as a (±) value.

Accuracy is the degree to which readings match an accepted standard (absolute) value and includes the combined effects of the sources of measurement error. Accuracy is written as a (±) value, such as ±1 mm, ±1% of reading, or ±1% of full span. The accuracy specification indicates that the measured value will conform to the absolute value within the stated (±) limits over the full range of specified operating conditions.

Precision is the closeness of approach of each of a number of similar measurements to the arithmetic mean. It is expressed as a (±) value. In dam monitoring precision is often more important than accuracy because change, rather than absolute value, is of greatest interest.

Repeatability is the closeness of agreement of a number of consecutive measurements to one another under the same operating conditions. It is expressed in engineering units, such as 1 mm, or as a percentage of full span. The difference between precision and repeatability is illustrated in Figure G-2.

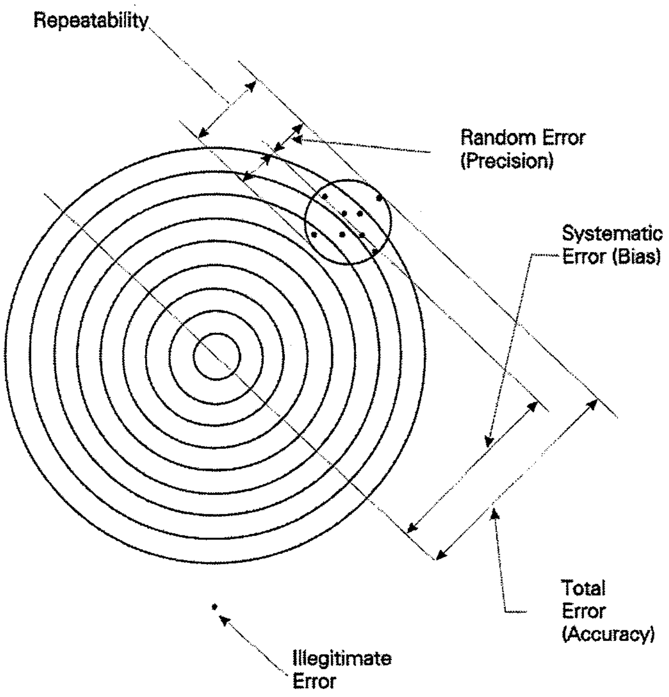

Figure 4.2 Accuracy terminology

Another important performance parameter, but one that is seldom specified by manufacturers or even consistently defined, is long-term stability. As defined for our purpose, long-term stability is an indication of the change in an instrument's output that would take place over a specified period of years in the absence of a change in the measured input (pressure, force, displacement, etc.). Because of the long time periods involved in gathering data and the diversity of factors that can contribute to output drift over these periods, precise answers can be elusive. However, publications discussing the long-term performance of most dam monitoring instruments are available and can be provided by the instrument manufacturers.

While instrument specifications define instrument performance, the total performance of a monitoring system is also a function of the readout or data acquisition equipment. For example, an instrument repeatability of 1 mm might be equivalent to an output repeatability of 1mV. If the readout or data logger is unable to resolve changes smaller than 2mV, the precision of the recorded data will be no better than 2 mm. Electronic noise picked up in long cables or radio links between the instrument and the recording device can further degrade performance. It is important to evaluate the combined specifications of instrument and data acquisition system, and to suppress noise pick-up, if optimal results are to be achieved.

5.0 Basic Instrumentation Concepts

A determination of the number, type, and location of instruments required at a dam or other earthen structure can only be addressed effectively by combining experience, common sense, and intuition. Embankments represent unique situations and require individual solutions to their instrumentation requirements. The instrumentation system design, therefore, needs to be conceived with care and consideration for the site-specific geotechnical conditions present in the embankment, foundation, abutments, and possibly the reservoir rim. Substantial geotechnical reasons, such as unique design or difficult foundation conditions, severe downstream hazard, visually observed problems or concerns, remoteness of location, normally unmanned operation, or other concerns justify providing instrumentation. It also must be recognized that instruments are actually discontinuities, or nonrepresentative objects, introduced into soil/rock structure systems. It is possible that their presence, or the flows or displacements required to generate an observation, could alter the parameters which are being measured. Engineers installing field instrumentation must understand the fundamental physics and mechanics involved, and how the various available instruments will perform under the conditions to which they will be subjected. An in depth discussion of the geotechnical considerations for District dams is presented in Appendix E.

6.0 Instrumentation System Planning

Planning an embankment instrumentation system requires the consideration of many factors, and a team effort of the designers (or those responsible for evaluating existing projects) and personnel having expertise in the application of geotechnical instrumentation. Developing an instrumentation plan should begin with a definition of an objective and proceed through a comprehensive series of logical steps that include all aspects that address dam safety. A series of recommended steps is provided below. This systematic planning is based on the approach recommended by Dunnicliff (1988). The steps are listed below in Table G-2.

Table G-2 Steps for Developing an Instrumentation System

Step	Element of Plan
a	Prediction of mechanisms that control behavior
b	Definition of purpose of instrumentation
c	Definition of geotechnical questions
d	Selection of parameters to monitor
e	Prediction of magnitudes of change
f	Selection of instrument locations
g	Selection of instruments
h	Determination of need for automation
i	Planning for recording of factors which influence measurements
j	Establishment of procedures for ensuring data validity
k	Determination of costs
l	Planning installation
m	Planning long-term protection
n	Planning regular calibration and maintenance
o	Planning data collection and management
p	Coordination of resources
q	Determination of life cycle costs

6.1 Purpose of Instrumentation

The primary purpose of instrumentation is to provide data useful for determining whether or not an embankment or foundation is behaving in accordance with engineering predictions. When an embankment has special foundation conditions or uncommon design features, instrumentation can help in determining if design concepts for these conditions are warranted. A comprehensive monitoring program must be developed for the special conditions and features at that site. If an embankment has no special foundation conditions or uncommon design features, the need for instrumentation is less certain. Many conventional embankments of moderate height on good foundations have only minimal instrumentation.

For most District dams, the materials selected for use in the upstream and downstream shells were determined in the field and may have been quite arbitrary. This is especially true for most District dams. These conditions are not necessarily bad, nor do they suggest future trouble. They merely reflect the design options available at the time of construction. Instrumentation, especially piezometers, is needed to identify and quantify piezometric differences with the different zones within the dam.

6.2 Location of Instrumentation

Locations for instruments should be determined based on predicted behavior of the site and consideration for possible failure scenarios. The locations should be compatible with the geotechnical concerns and the method of analysis that will be used when interpreting the data. A practical approach to selecting instrument locations for earth dams includes:

- Identify zones where problems may have occurred during construction;
- Identify areas that are most heavily loaded;
- Select zones that can be represented by typical cross sections where predicted behavior is considered representative of behavior as a whole;
- Install additional instruments at other potentially critical secondary locations such as in the abutments to determine flow conditions around and under the principal embankment section.

Although survey monuments are part of the instrumentation design, they have not been included in this discussion because all of the District dams already have extensive survey monuments installed. As part of the Dam Safety Program, a proposal will be developed for consideration in which permanent GPS stations will be placed at several District dams to determine the effectiveness of using such techniques for dam monitoring.

Damage during construction to the instrument, riser, or cable should be prevented by good design and temporary and permanent protection of surface exposure. Protection against vandalism should also be part of the design.

The preliminary instrumentation design for the ten dams requiring additional instrumentation is presented in Appendix H.

6.3 Selection of Instrument Types

The District has a long-history of successful operation and performance of their dams. The current problem is that there is a gross lack of reliable instrumentation at any of the dams, excluding Lenihan Dam. As further discussed in Appendix G, which presents the preliminary instrumentation design for ten District dams, it has been determined that only two major instruments are needed to significantly elevate our understanding of future dam performance. These include the vibrating gauge (electric) piezometers and the electric (automated) inclinometer. Due to the high probability of future earthquake shaking within a reasonable distance of our dams, we also propose to install accelerometers at several dams to not only capture the earthquake accelerations on the crest of the dam, but also act as triggering mechanisms for more rapid recovery of data in an attempt to capture earthquake induced pore pressures and deformations.

6.4 Determination of Need for Automation

Automated data acquisition systems (ADAS) have become a valuable means of collecting geotechnical instrumentation data. Developments in the field of electronics have made it possible to install and operate remote ADAS that provide accurate, reliable, and effective real time data collection. With increased emphasis on dam safety and the continued decrease in manpower, the advantages of providing automatic systems are numerous. Consideration should be given to providing ADAS at new projects and to retrofitting at existing instrumentation projects. However, it must be understood that while ADAS can provide valuable data in a timely manner, they are only to be considered as an integral part of an overall dam safety program. An ADAS does not replace the need for detailed visual inspections of the dam on a regular basis. An in-depth discussion on ADAS is presented in Appendix F.

6.5 Determination of Instrumentation Costs

When the need for instrumentation is properly and correctly established, and when the program is properly planned, there should be sufficient justification for funding. Instrumentation for embankment dams does not have to reduce costs to be justified. If the design is appropriate, and the system is properly maintained, a modern instrumentation system can be valuable in proving, on a day-to-day basis, that a dam is safe. When evaluating the cost of a new instrumentation system, the value of ensured safety and the avoidance of failure, saving the cost of damages and/or cost of major repairs, makes instrumentation programs cost-effective. Cost estimates should take into consideration the cost of instruments, installation, calibration, automation, long-term protection, maintenance, and data collection and management.

In developing the cost estimate for the DIP, special emphasis was placed on the actual costs incurred in completing the instrumentation at Lenihan Dam. It turns out that there are several methods that can be developed based on such variables as the total number of instruments, total number of drill holes, the average depth of the instruments, or combinations of these variables. In the end, judgment and experience were used to estimate the basis of cost which varied from dam to dam, depending on the number and location of the planned instruments.

7.0 References

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APPENDIX H

PRELIMINARY INSTRUMENTATION DESIGN FOR ALMADEN, ANDERSON, CALERO, CHESBRO, COYOTE, GUADALUPE, RINCONADA TREATED WATER RESERVOIR, STEVENS CREEK, UVAS AND VASONA DAMS

APPENDIX H

PRELIMINARY INSTRUMENTATION DESIGN

1.0 Introduction

This appendix presents the preliminary instrumentation design for ten (10) dams that are owned and operated by the Santa Clara Valley Water District. The preliminary instrumentation design was carried out under the Dam Instrumentation Project project, which is an integral part of the overall Dam Safety Program. With regard to the instrumentation design for the ten dams presented herein, it is estimated that the process of final design, development of plans and specs, and construction, will take about 5-6 years to fully implement.

The nature and type of instrumentation discussed herein has been selected because of its effectiveness, overall reliability, and potential for automation. The installation of an automated instrumentation system was finally completed at Lenihan Dam in 2001. The instrumentation at Lenihan Dam, which is further discussed in Section 2.0 below, formed the model of the preliminary instrumentation at the other ten (10) District dams discussed herein. With the completion of the preliminary instrumentation design for the other ten (10) dams under the Dam Instrumentation Project (DIP), the District committed to send these designs to DSOD for their preliminary review and comments. This was accomplished in July 2002. Prior to submitting the final DSP to senior management for review and approval, it was also decided to have the DIP reviewed by an independent consultant. This review is currently being completed by GEI Consultants and their draft report is expected in March 2003.

The District will begin the environmental review, assessment, and CEQA documentation associated with the planned instrumentation program in April 2003. The final DIP will include detailed instrumentation layouts for each dam, an automation strategy, a CEQA document, and will serve as a decision document/Engineers Report for approval by District management. We currently envision that the final DIP work will be completed by the District's Capital Program Services Division that will be started in FY2004 upon completion and acceptance of the Dam Instrumentation Project decision document/Engineer's report by mid-summer 2003, depending upon CEQA process. The District currently envisions completing the instrumentation installation described in this report under several contracting efforts, each one involving perhaps two to four dams and being completed sequentially as shown in the prioritization schedule presented in Figure H-1. The dates presented in Figure H-1 are changed slightly from the schedule originally provided to DSOD, especially for Anderson, Uvas, and Rinconada Dams. As shown in Figure H-1, we estimate that the instrumentation installation of first series of two (2) dams, Almaden and Guadalupe Dams, will be completed by the end of 2004 and that the final series of two (2) dams, Vasona and Rinconada Dams, will be completed by 2008.

2.0 History of Instrumentation at District Dams

All District dams were nominally instrumented when they were originally constructed back in the 1930's or 1950's. Each dam had a series of survey monuments installed (typically at 100-ft stations) on the dam crest across the entire width of the dam. Typically, monuments were also installed in both abutments that were assumed to be fixed in bedrock. In some instances, such as at Anderson Dam, as the dam settled and the roadway over the dam was repaved, the surface monuments had to be reset. In general, however, the surface monuments at all of the dams were sturdy, well secured and read with a frequency such that it was possible to reconstruct both the lateral and vertical movement experienced by the dam crest back to the time of construction. This information has been very helpful, especially with regard to determining the approximate crest movement as a result of the 1984 Morgan Hill earthquake (M=6.2) and the 1989 Loma Prieta earthquake (M=6.9). These movements have been periodically documented and reported to DSOD in the annual surveillance reports prepared by the District for each dam.

The only dam for which horizontal control could not be adequately reconstructed was Coyote Dam. The dam was built across the active Calaveras fault, which is located in the left (west) abutment of the dam. Since the horizontal control line for the dam crest crossed the fault, which is currently moving at a rate of about 14-15 mm/year, one end of the base line was always moving, thereby negating the ability to use station and offset survey methods to determine movement of the base line. In 1995, the District began to use the Global Positioning Satellite (GPS) method to establish Northing and Easting locations of monument points at Coyote Dam. This method has proved to be very reliable in monitoring movements at the dam, especially if tectonic movements are thought to be occurring. GPS methods are fast becoming the survey method of choice for horizontal control of dam monuments. The District is currently evaluating establishing permanent GPS survey equipment at several of its facilities. Clearly, the GPS survey method has a prominent position in the future surveillance of District dams.

The history of installing piezometers in District dams dates back to the mid 1970s. In 1975, eight (8) pneumatic piezometers were installed in the upstream shell of Guadalupe Dam due to the concerns of incipient failure due to drawdown. Based on the results of this instrumentation and the resulting stability analyses, a major buttress was placed over the lower portion of the downstream shell. The original pneumatic piezometers at Guadalupe Dam subsequently failed and have been abandoned. Also in 1975, during the initial safety investigations of Anderson and Coyote dams, pneumatic piezometers were installed within the impervious core of each dam. These instruments are still active and functioning. Both open well and pneumatic piezometers were subsequently installed at other District dams during the 1980s as part of safety investigations; however, many of them have either failed or been inadvertently destroyed during subsequent maintenance or construction work performed on the dams. The two principal exceptions have been Uvas and Stevens Creek Dams. At Uvas Dam, constructed in 1957 by the Gavilan Water District (GWD) and taken over by the District following their merger with GWD in 1987, 36 open well piezometers have been installed in an attempt to monitor water surface elevations as a result of seepage problems in both abutments and the embankment. A major grouting effort in 1996 in the right abutment area appears to have significantly curtailed the seepage occurring through this abutment. At Stevens Creek Dam, seven (7) pneumatic piezometers were installed in 1976 during an initial safety investigation. All but two of these instruments were destroyed during the 1985 seismic retrofit work that added major stabilizing buttresses over both the upstream and downstream shells. During the seismic retrofit work, nine (9) new pneumatic piezometers were installed within the central and downstream shell. These instruments are still active and functioning and are read on a monthly basis.

A major event occurred at Lenihan Dam in 1998 that vividly brought to light the need for modern instrumentation for all District dams. The 50-inch diameter outlet pipe, which had experienced several collapses (internal buckling of the 5/16-thick steel liner) in the past, was being investigated when it was discovered that unusually high pore pressures were present adjacent to the outlet pipe over much of its length, especially through the impervious core section of the embankment. Due to the lack of adequate instrumentation, it was not known how extensive the higher than expected pore pressures were, whether they were unique to the outlet pipe area, or even when the high pore pressures had been initiated. When it was discovered that the actual pore pressures were higher than those predicted by seepage modeling, it was decided that there was a distinct possibility that internal piping of the dam could either be initiated, or perhaps already been propagated adjacent to the outlet pipe. The District declared a Level One Emergency and immediately began an investigation to determine the probable cause and extent of the higher than expected pore pressures. A full and complete description of this work, in addition to the types and locations of geotechnical instruments installed during the investigation and subsequently, is presented in various reports prepared for the District (RLVA, 1999; SCVWD, 2001).

Suffice it to say that Lenihan Dam has become one of the most heavily instrumented embankment dams in the United States. In total, 22 vibrating gauge piezometers were installed at various locations adjacent to the outlet pipe, 25 piezometers were installed at various locations within the upstream and downstream shells, and 7 piezometers were installed in the foundation bedrock in the upstream and downstream right abutment. In addition, two inclinometers were installed from the dam crest and the approximate mid-height of the dam. These inclinometers were automated by installing a total of 12 bi-axial tiltmeters using an innovative design developed for Lenihan Dam. In addition to the piezometers and inclinometers, a strong motion accelerometer was installed and connected to the two measurement and control units (MCU) that made up the automated data gathering system. The MCU's were programmed to take readings of all instruments four times per day at 6-hr. increments. The accelerometer is set such that if the acceleration sensed by the unit ever exceeds 0.05 g, then the MCU's are programmed to begin taking data of all instruments every 15 minutes until programmed to stop. Hopefully, this will facilitate developing an idea of the magnitude and duration of earthquake-induced pore pressures and displacements that can develop within the dam. The success of the instrumentation, and the automated data gathering system and computerized presentation of results, has formed a valuable model on which to complete the instrumentation at other District dams.

It should be noted that one issue that remains to be resolved at Lenihan Dam has to do with the need to monitor potential seepage near the downstream toe. DSOD has requested that the District remove existing brush covering the downstream toe area in order to be able to more easily observe possible seepage. Significant CEQA and ESA issues will delay the required cleanup. Although there has never been any reported seepage in the downstream toe area, the District will continue to monitor the new instrumentation installed in the downstream embankment, as well as carefully monitor for emerging seepage once the brush area has been cleared. As discussed in previous reports (RLVA, 1999), a major portion of the downstream shell is underlain by a massive horizontal drain that extends up to elev. 483 (NAVD, 1988). The District does not consider additional seepage control measures necessary or warranted at this time. In the event that future monitoring or observations suggest the possibility of emerging seepage, the District will undertake appropriate actions at that time.

3.0 Preliminary Design of New Instrumentation

The piezometer type selected for use in Lenihan Dam was the vibrating wire gauge piezometer. This instrument was selected because of its robust character, very small diaphragm movement for accurate readings in impervious materials, and its ability for automation. Two different methods were used in installing the vibrating gauge piezometers at Lenihan Dam. The first method, and by far the most often used, involved the conventional method of rotary wash drilling followed by cleansing of the hole, installation of a nominal thickness of fine sand surrounding the piezometer tip, and then grouting (sealing) the remainder of the hole to prevent intrusion of surface water, or intermingling of pore pressures from other areas of the dam through the drill hole. The second method, which was the fastest and cheapest, involved the installation of a push piezometer (also vibrating gauge type) by use of a Cone Penetrometer Test (CPT) rig. In the later method, two techniques (both successful) were used to protect the leading edge of the piezometer (where the electronic gauge is housed) from being over-stressed during installation. The first method involved the use of a sacrificial "bonnet" or shield that was placed in front of the piezometer. The second, and perhaps more foolproof, involved the use of a regular cone that was used to advance a "pilot hole" down to the desired elevation of the instrument. The regular cone was then withdrawn and the piezometer tip was then advanced to the desired elevation with little or no resistance. With both techniques, a nominal amount of sand was poured down the hole to provide a sensing window above the piezometer tip and grout was injected into the approximately 1-inch diameter hole as the rod was pulled to prevent surface water from having an easy path down to the tip. In general, the time required for installing a piezometer using the conventional rotary wash method averages between 1-2 days. Using a CPT rig, it is possible to install 3-4 piezometers per day assuming relatively easy driving access to the site by the CPT rig. Four push piezometers were installed at Lenihan Dam; they have continued to respond in the same manner as other conventionally installed piezometers.

The new instrumentation will be installed using conventional drilling techniques, either rotary wash or hollow stem auger methods as appropriate. The District would like to install several push piezometers using the CPT rig where it would be economically viable and appropriate, and will discuss the pros and cons of this possibility with DSOD prior to the development of final design. Also, the District considered the possibility of using directional drilling to install some of the piezometers. Directional drilling would be ideal especially if the drill rig could set up on the crest and install piezometers both upstream and downstream from the crest. After careful consideration, however, the possibility of using directional drilling was dropped when it became obvious that such a method could introduce major concerns during piezometer installation, such as 1) the possibility of hydraulically fracturing the core due to the need to introduce higher fluid pressures to control the direction of the drill head, and 2) not knowing the exact final location and elevation of the piezometer. It was decided that positioning of the upstream and downstream shell piezometers could be successfully located by performing adequate site preparation work (e.g. building drilling pad). For example, many of the District dams have concrete facing instead of riprap to protect against erosion of the upstream slope due to wave action. Consequently, to successfully install piezometers within the upstream shell, it will be necessary to cut out an access window (about 10' x 10') in the concrete for the drill rig and cut a smaller access trench up the face of the concrete to the crest for the piezometer wiring. The concrete facing would be restored once the piezometers have been installed. Similarly, many of the downstream faces are covered with coarse rock rubble, which generally came from oversize material created during excavation of the spillway. For drilling access and to construct a platform area it will be necessary to temporarily remove these materials.

The results of the preliminary design for Almaden, Anderson, Calero, Coyote, Chesbro, Guadalupe, Rinconada Treated Water Reservoir, Stevens Creek, Uvas, and Vasona Dams are discussed in the following sections of this appendix. These designs are currently being peer reviewed and the results of this peer review will be incorporated into the final DIP final design. The preliminary design discussed herein focuses on the location of piezometers. In all instances, the recommended piezometer type will be the electronic vibrating gauge piezometer. In addition to the new piezometers, as discussed in more detail below, instrumentation at each dam may also include inclinometers, accelerometers, a computerized data collection system, and the construction of a seepage monitoring and collection system. It will be noted that the number of new instruments varies from dam to dam. Although this is partially due to dam height, the number of new instruments is more governed by the history of seepage problems that have occurred at each dam.

The installation of the new instrumentation at each dam will afford a unique opportunity to obtain undisturbed samples of the embankment and foundation materials, and to core the underlying bedrock materials. Typically, an undisturbed sample is taken at the tip elevation of each piezometer in order to determine the engineering properties of the surrounding materials. Similar work completed at Lenihan Dam during the installation of the instrumentation in both the upstream and downstream shells provided valuable reference data regarding the engineering properties of the embankment soils. Embankment and foundation samples will be tested in the laboratory to determine the geotechnical engineering properties related to compressibility, permeability, and shear strength of the tested materials. In addition to recovery of bedrock core, field water pressure tests will be performed at regular intervals in the bedrock area to enable an assessment of *insitu* permeability of the native bedrock materials.

The type of instrumentation planned for all District dams has been selected because of its robustness, accuracy and its ability for automation. Although the automation design details are not discussed herein, equipment design similar to that currently being used at Lenihan Dam has been described in a recently published technical paper (Volpe, Nelson & Schuyler, 2000). The District envisions hiring an instrumentation consultant to provide assistance with the design of the ultimate system to be installed for all eleven dams (including Lenihan Dam). The final system will provide daily readings of all instruments and will provide the ability to easily document trends in data results. It should be noted that two of the dams discussed herein, Rinconada and Vasona Dams, may not need to be hooked up for daily automation. These two smaller dams will be provided with new electronic piezometers like other dams, but it is likely they will be hooked up to data loggers that can be programmed to collect data on a daily basis but may only be downloaded on a monthly basis.

Subsequent to completing the instrumentation at Lenihan Dam, the District has been in discussions and meetings with the California Division of Mines and Geology (CDMG) and the U. S. Geological Survey (USGS) regarding the installation and future reading of strong motion accelerometers at various District dams as part of the Dam Safety Program. CDMG, Caltech, USGS, and U.C. Berkeley have joined forces to install new monitoring equipment and develop new computer programs to accurately monitor potential strong ground shaking throughout California. The fundamental goal of the program is to get better coverage for their efforts in developing instrumental intensity maps very quickly after major earthquakes. The District and CDMG have joined forces to install several additional strong motion accelerometers downstream of existing dams, but sufficiently close that they can be connected to existing phone lines and buried instrumentation piping. The new instruments will be installed on District property by CDMG with help from the District with site preparation. The District is also joining forces with USGS with strong motion instruments to be installed by the District as part of the Dam Instrumentation Project. The USGS has extensive experience and specialized computer programs to analyze and interpret the accelerograms that are generated by the strong motion instruments during earthquakes. USGS has offered to assist the District with the collection and interpretation of any District-owned accelerograph that is a part of the Dam Instrumentation Project. This will provide for a more accurate interpretation of the collected data and, most importantly, provide the USGS with more data related to critical structures in the event of an earthquake. The District will keep DSOD informed as this cooperative effort goes forward.

3.1 Almaden Dam

3.1.1 Piezometer Requirements

The proposed locations and elevations of the new piezometer instrumentation for Almaden Dam are presented in plan and section in Figures H-2 and H-3 and summarized on Table H-1, respectively. To avoid confusion between similar numbered piezometers at different dams all new instrumentation will be identified and designated using the unique State Dam Number for each dam. As shown in the two referenced figures, twelve (12) vibrating gauge piezometers are planned to provide the necessary information on pore pressures acting within the embankment and foundation. Piezometers to be installed within the upstream shell will have to be installed through access windows cut in the concrete facing that protect the upstream dam face. In developing the plans and specifications for the installation of the instrumentation for all of the dams discussed herein, it should be noted that some of the piezometer locations on the upstream side of the dam could be inundated during full reservoir conditions. The work associated with the installation of these piezometers should be planned to be performed during the late summer to early fall season when the likelihood of reservoir filling is lowest.

In previous discussions with DSOD, it has been agreed that nesting of piezometers (more than one piezometer per drill hole) will only be allowed for embankment piezometers and not for piezometers in the foundation and the embankment. Keeping embankment and foundation piezometers separated in different drill holes significantly lowers the likelihood of interference between these two separate and distinct hydrologic units. It is estimated that 9 drill holes will be required to install the 12 new piezometers.

The preliminary design also provides for installing a piezometer to monitor pressures within the downstream left abutment in the area of the recently discovered fillet fill that was apparently placed over the downstream shell immediately following construction. As shown in Figure H-3, one piezometer (72-4-11) is planned at STA 3+86 about 50 feet upstream of the dam axis to monitor pore pressures acting within the foundation near the area where problems associated with the silica carbonate rock (see Section 3.1.2) were encountered during dam construction. A second piezometer is planned about 170 feet downstream of the axis to monitor pore pressures acting between the embankment and the fillet fill.

In addition to the new piezometers discussed above, the preliminary design also includes the installation of one or more automated inclinometers and one strong motion accelerometer to act as a trigger to collect more data in the event of earthquake shaking. Details regarding these instruments will be developed during final instrumentation design.

3.1.2 Seepage Collection Requirements

Almaden Dam has had a history of recurrent seepage at the downstream toe area, especially emanating from the left abutment area in contact with the fillet fill that was placed over the downstream shell following construction. Marliave logged a large mass of serpentinized ultramafic rocks (serpentinite) in the left abutment, and thin zones of serpentinite were logged along the outlet pipe. Silica carbonate rock was encountered in the left abutment during excavation of the cut-off trench. Serpentinite masses are believed to have formed as the result of thrusting of oceanic crust into Franciscan Complex rocks during abduction of these rocks onto the North American Continent during the Jurassic. Silica carbonate was formed by hydrothermal alteration of serpentinite, probably during the Miocene.

The occurrence of silica carbonate rock in the left abutment during excavation of the cut-off trench presented a significant challenge with respect to constructing the cut-off trench in a suitable foundation. It is now apparent that the presence of the silica carbonate material encountered in the left abutment side of the cutoff trench was perhaps the principal reason for placement of a fillet fill over a portion of the downstream shell following construction. Silica carbonate is typically full of large voids due to removal and alteration of serpentinite minerals during hydrothermal alteration. Marliave (1936) states that "*the rock is badly crushed and honeycombed and is not uniform in composition or character. It may be soft or hard and contains irregular open cracks. It contains silica skeletons and carbonate segregations with much clay and numerous soft limonite pockets. It is treacherous rock in which to land a cut-off for a dam.*" Consequently, the original alignment of the cut-off trench had to be altered during construction so that it would terminate into more favorable serpentinite bedrock.

Other indications that the foundation materials were locally marginal include the following geologic features described by Marliave (1936). Serpentinite encountered in the cut-off trench *was "badly sheared and soft. Several springs were encountered, two of which are noted in the geologic section….. These springs were not deep-seated but were fed by locally absorbed water* *on the hillside; still it was necessary to drain them or to seal them off."* The as-built drawings contain notes where springs were encountered during excavation of the cutoff trench or the foundation. One note states… "Pipes in Cut-off Trench to tap springs." In addition, a French drain approximately 200 feet long was constructed directly east of the outlet pipe through the downstream toe of the dam.

In order to deal with the seepage issues described above, a new seepage collection system is planned for the downstream toe area. We anticipate that construction of the seepage collection system will involve excavating a section of the downstream toe area in order to expose sound bedrock and embankment materials, and installing the necessary drain pipes, drain materials, and appropriate geotextile fabric to adequately collect and discharge the collected seepage. In lieu of conventional seepage monitoring equipment (e.g., "V" notch weir), the District is considering the installation of automated seepage weirs at Almaden Dam. The necessary design for the seepage collection system will, most likely, be completed and submitted to DSOD with the preliminary plans and specifications for the instrumentation application.

3.2 Anderson Dam

The proposed locations and elevations of the twenty (20) new piezometers for Anderson Dam are presented in plan and section in Figures H-4 and H-5 and summarized on Table H-2, respectively. As noted previously, Anderson Dam was one of the first dams to be instrumented by the District and the two pneumatic piezometers that were installed in 1975 are still functioning. However, significantly more piezometric information is required, especially within the foundation, to assess the safe dissipation of seepage pressures through the foundation. The primary focus of the new instrumentation is the expanded focus on the impervious core and underlying foundation bedrock sections. Because the upstream and downstream shells of the dam are comprised of rockfill materials, it would be very difficult and costly to try and instrument the rockfill zones. Furthermore, it has been assumed for purposes of dam stability that the entire reservoir head passes unimpeded through the upstream rockfill zone and acts directly against the upstream side of the core (i.e., the interface of the core and upstream shell zones). For stability purposes, it was assumed that the entire hydraulic head imposed by the reservoir is dissipated as seepage losses through the impervious core. In other words, even though the rockfill may be considered "dirty", it is tacitly assumed that the rockfill zones are relatively free draining, especially when compared to the impervious core. The new instrumentation is designed to help confirm this assumption.

The principal instrumentation section of the impervious core will be located at STA 8+50, near the maximum dam section. We have determined that the most ideal location for these new piezometers is immediately beneath the upstream crest of the dam. In order to accomplish this, we anticipate removing the existing guardrail on the upstream side of the crest and constructing a drilling pad so that the drill rig can be properly located over the upstream crest. Two drill holes will be required to install the four piezometers shown. Other piezometers will be installed with the drill rig located near the center and downstream crest location. As shown in Figure H-5, the line of instruments within the core will be vertically spaced immediately beneath the intersection of the upstream slope and dam crest. As shown in Figure H-5, the additional piezometers are equally scattered within the impervious core and underlying bedrock throughout the left, central, and right abutments of the dam.

In total, there are thirteen (13) piezometers planned for the impervious core and seven (7) piezometers planned for the foundation. The purpose of piezometer 72-9-20 is to provide piezometric data from the bedrock near the spillway entrance area. New seepage collection weirs were recently (2000) installed at the base of the spillway. These weirs monitor the discharge from an array of underdrains located beneath the spillway slab for both the original and new spillway sections. Based on previous response of the two weirs, it appears that the volume of weir flow significantly increases when the reservoir level exceeds about elev. 618 feet. This piezometer will help monitor changes in foundation seepage conditions in the area of the spillway.

We estimate that 14 drill holes will be needed to install the 20 new piezometers planned for Anderson Dam. The holes will total about 2,215 feet through the embankment and 270 feet into underlying bedrock.

The District is also planning to install an observation well near the downstream toe (see Figure H-4 for location). The observation well, which will be about 18-24 inches in diameter, will be finished with a hard plastic (PVC or equivalent) casing with a lockable cap. We estimate that the depth to bedrock for this hole will be less than about 20 feet. The observation hole will penetrate bedrock but be finished off as an observation well with slotted grooves and geotextile protection to enable ground water/seepage to easily enter the well. Since it is tacitly assumed that the groundwater conditions at this location are highly influenced by seepage from the dam, this observation well will enable the monitoring of groundwater level for various reservoir operations and an assessment of corresponding seepage conditions. Sampling of the well for water quality will be possible.

In addition to the new piezometers discussed above, the preliminary design will include the installation of one or more automated inclinometers and one strong motion accelerometer to act as a trigger to collect more data in the event of earthquake shaking. The scope of this preliminary design work does not include further discussion of these additional instruments. Details regarding these instruments will be developed during final instrumentation design. With the installation of the two new seepage collection weirs near the base of the spillway in 2000, no other seepage collection instruments are considered necessary for Anderson Dam.

3.3 Calero Dam

3.3.1 Piezometer Requirements

The proposed locations and elevations of the twenty-two (22) new piezometers for Calero Main Dam are presented in plan and section in Figures H-6 and H-7 and summarized on Table H-3, respectively. No new instrumentation is planned for the auxiliary dam (See Section 3.3.4) or for Fellows Dike (See Section 3.3.5). Piezometers to be installed within the upstream shell will have to be installed through access windows cut in the concrete facing protecting the upstream face.

In addition to the typical upstream and downstream locations for piezometers shown in Figure H-6, the preliminary design involves installing piezometers in the right abutment area to evaluate the foundation and embankment conditions that are possibly contributing to seepage in this area, as discussed below. In total, it is estimated that 15 drill holes will be required to install the 22 new piezometers.

The preliminary design will also include the installation of one or more automated inclinometers and one strong motion accelerometer to act as a trigger to collect more data in the event of earthquake shaking. As stated above, details regarding these instruments will be developed during final instrumentation design.

3.3.2 Right Abutment Construction Issues

Leakage through the right abutment has been an issue at the dam since the end of construction and first filling of the reservoir. Construction problems had occurred during construction and were generally resolved as follows. After the foundation of the main dam was stripped of alluvium and highly weathered rock, a cutoff trench, typically 15 feet deep and between 10 and 20 feet wide at its base, was excavated beneath the impervious zone. Due to problems in finding suitable foundation materials, the cutoff trench in the upper part of the right abutment was abandoned and an alternative trench was excavated upstream of the original one, beginning at about elevation 420 feet (NGVD, 1929). In the alternative trench, less competent foundation materials were encountered in a suspected fault zone in the vicinity of the groin contact of the right abutment. Consequently, the cutoff was excavated deeper (as much as 45 feet) and a 38 feet-deep sheet pile wall was installed below the trench between stations 1+63 and 2+01. The average depth of the piling was 14 to 15 feet and it extended 5 feet above the trench bottom into the embankment (DWR, 1981). Below elevation 463 feet (NGVD 1929), the cutoff was excavated into relatively intact Franciscan Complex bedrock; above 463 feet, the cutoff did not penetrate entirely through the highly fractured intrusive rock.

The cutoff trench backfill and the embankment fill materials were poorly compacted to about elevation 447 feet (NGVD 1929). The DSOD inspector recognized this deficiency in a pit during construction but decided that, "considered solely from a standpoint of safety, the degree and magnitude of the condition did not merit disapproval of the unsatisfactory portion of the fill" (Wahler Assoc., 1981). The remainder of the embankment was constructed in a satisfactory manner.

3.3.3 Weirs – General Description and History

Two weirs have been used for measuring fluctuations in seepage flow rates from the right abutment of the main dam. The weirs are triangular 90 notched weirs with end contractions. Findings from an evaluation of the weirs by a District engineering geologist indicated the existing weirs are of "nonstandard design" (Stejer, 1995). Therefore, these weirs do not provide accurate seepage flow rates.

Initially, two temporary weirs were installed in 1965, near the downstream base of the right abutment. In July 1967, these weirs were replaced by steel triangular notched weirs installed in concrete abutments (Roman and Wire, 1969). Two generally southwest-trending concrete-lined channels were installed directly upslope of the weirs in the right groin area as shown on Figure H-6. These channels run roughly parallel to the local contour so that seepage flowing downslope is intercepted and directed to the two weirs. The channel upslope of weir W1 is lined with concrete along the southwestern portion for about 110 feet and is unlined along the northeastern portion for about 20 feet. Weir W1 is connected to the main channel by a shorter concrete-lined channel that is approximately 17 feet long and runs roughly perpendicular to the main channel. The channel draining to weir W2 is approximately 40 feet long, with the weir at the southwestern end of the channel.

In 1991, Wahler Associates installed three 2-inch diameter PVC pipes in the right groin area of the main dam at about elevation 450 feet (NGVD, 1929), to temporarily measure seepage during the 1991 right abutment grouting program. Seepage data from these pipes, designated as "temporary seepage measuring point", were used by Wahler Associates to evaluate the effectiveness of the grouting program while the work was in progress. Two of the temporary seepage pipes are embedded in fractured bedrock directly upslope of piezometer C-3. The third pipe, located several feet downslope of the first two pipes, is connected to two galvanized sheet metal plates that act as a coffer dam to collect upslope seepage runoff and direct it to the upslope end of the third pipe. The District has not routinely collected seepage data from the temporary seepage pipes. However, seepage data were collected from these pipes in April 1995 and June 1996. On April 19, 1995, flows from the two upslope pipes were estimated as no flow to a trace, with about 14 gallons per minute (gpm) flowing from the third downslope pipe (Stejer, 1995). On June 29, 1996, flows from the two upslope pipes were estimated as no flow to a trace, with about 7 gpm flowing from the third downslope pipe.

It should be noted that in January 2001, after inspecting the area, the District determined that the seepage collection and distribution system located in the downstream right abutment groin area had been badly damaged by rooting wild boar. Due to the extensive damage to the entire seepage collection system, it was no longer possible to continue reading the weirs.

In order to deal with the seepage issues in the right abutment described above, a totally new seepage collection system is planned for the entire right abutment and downstream toe area. We anticipate that construction of the new seepage collection system will include excavating and exposing firm bedrock and embankment materials of the entire downstream right abutment area, and installing the necessary drain pipes, drain materials, and appropriate geotextile fabric, to adequately collect and discharge the collected seepage. The District is currently (February 2003) evaluating a soil/bedrock exploration program to develop the necessary design parameters, dimensions, and lateral extent of the required seepage protection facilities. The necessary design for the seepage collection system will, most likely, be completed and submitted to DSOD with the preliminary plans and specifications for the instrumentation application.

3.3.4 Auxiliary Dam

The saddle dam at Calero Dam is located about 3,500 feet east of the main dam, is about 40 feet high, and has a crest length of about 500 feet. The saddle dam, constructed in 1935 at the same time as the main dam, was necessary to contain the reservoir from flowing into a small drainage adjacent to the new alignment of McKean Road. The saddle dam has been instrumented with surface survey monuments and five (5) open well piezometers. These instruments are still operational and the dam appears to be performing satisfactorily with respect to seepage. Consequently, there does not appear to be any justification for installing new instrumentation in the saddle dam and, therefore, none is recommended at this time.

3.3.5 Fellows Dike

The Fellows Ranch Dike is located at the extreme southern end of Calero Reservoir. The dike serves two purposes: 1) it controls the flow of Calero Creek into the south end of the reservoir; and 2) it protects the historical Fellows Mansion and adjacent Calero Lake Park and riding stables from being flooded during periods of high reservoir conditions. The dike was constructed in 1935-36 during construction of the dam by a contractor other than the main contractor. The original alignment of McKean Road was relocated around the east side of the reservoir to prevent inundation. The dike was constructed to an unknown set of specifications by the contractor who built the new McKean Road alignment. Other improvements to the dike occurred at various times since the original construction. Today, the dike, which parallels Calero Creek where it empties into the reservoir and then turns 90 degrees west to tie into the adjacent hillside, prevents flooding of the old Fellow's mansion and a horse ranch and stable area. The dike is about 2000 feet-long and has a crest at about elevation 490 feet (NAVD 1988). It has a maximum height of about 15 feet with irregular side slope inclinations.

During the 2000 safety inspection, DSOD indicated that they consider the Fellows Dike as a jurisdictional dam and, as such, requested that the District complete an in-depth safety investigation of the dike. Since that time, the District had a study completed by GEI (2001) that involved detailed seepage, stability and earthquake stability analyses of the dike. The final study report was submitted to DSOD in November, 2001. The District is currently developing final plans and specs for the modification at Fellows Dike. We envision final construction to be completed in 2004.

As a part of the detailed geotechnical investigation, two open well piezometers were installed in the dike. The District plans to continue reading these instruments on a monthly basis as a part of its instrumentation monitoring program. No new instruments are currently envisioned for Fellows Dike at this time.

3.4 Chesbro Dam

Chesbro Dam was constructed in 1955 by Gavilan Water District as a zoned earthfill dam with an impervious zone in the upstream half and a pervious zone in the downstream half. Zoning within the dam was achieved by placing relatively finer-grained materials upstream in an impervious zone and coarser-grained materials in the pervious zone downstream. Since the zoning was achieved by selective routing of construction hauling equipment within a single borrow area for the embankment fill, the distinction is slight between the grain size distribution in the impervious zone and in the pervious zone. Specifically, the embankment consists of homogeneous stiff, brownish gray clayey sand with gravel. A compacted fill buttress was constructed at the downstream toe. A 4-foot thick gravel blanket drain lies between the rolled earth fill and the compacted fill buttress.

A cut off trench was constructed slightly upstream of the midpoint of the "impervious" zone. The cut off trench was excavated across the channel floor and up both abutments. Prior to grouting operations the trench was excavated a few feet short of final grade. Grouting consisted of a single line of grout holes placed on 5- to 10-foot centers in the central axis of the trench, with a maximum grout depth of about 80 feet. After grouting, about 3 to 4 feet of native materials were removed from the cut off trench before compaction of the embankment was started.

The downstream half of the dam was constructed on unconsolidated alluvial gravel and sand deposits that were left in place. These deposits were up to about 10 feet thick. In addition, construction records describe the presence of "older alluvium" in the stream channel that was up to about 40 feet thick that consisted of boulders of serpentinite in a matrix of gravelly material. It is not clear whether this was actually an alluvial deposit or just extremely weathered serpentinite mélange.

The foundation seepage is measured from a drain pipe that runs from the dam foundation, through the northern spillway wall, and into the lower end of the spillway. The pipe is in contact with a small blanket drain constructed beneath the embankment to intercept seepage from in-place pervious gravels that lie across the valley floor. The maximum reported seepage flows were 35.0 gallons per minute (gpm) on February 24, 1998. This magnitude of seepage suggests that seepage losses at Chesbro Dam are not a major concern.

The proposed locations and elevations of the sixteen (16) new piezometers for Chesbro Dam are presented in plan and section in Figures H-8 and H-9 and summarized on Table H-4, respectively. As shown in Figure H-8, the preliminary design specifies that piezometers will be installed along two essentially similar sections of the dam; one in the central part of the valley near the maximum dam section at STA 8+00 (Section A-A'), and one on the right (west) side of the valley at STA 6+50 (Section B-B') on the left side. In addition to developing piezometric conditions throughout the dam, one of the principal goals of the instrumentation design is to determine whether a different piezometric head is acting within the downstream gravel alluvium that was left in place. In total, it is estimated that 12 drill holes will be required to install the 16 new piezometers.

In addition to the new piezometers discussed above, the preliminary design will include the installation of one or more automated inclinometers and one strong motion accelerometer to act as a trigger to collect more data in the event of earthquake shaking. Details regarding these instruments will be developed during final instrumentation design.

To our knowledge, no seepage issues have ever been raised at Chesbro Dam. The functional operation of the existing seepage collection system indicates that the collection system and drains are working properly. The new instrumentation will provide additional information in the future regarding the piezometric conditions within the downstream shell and toe area. Based on the results of this future monitoring, an assessment will be made regarding whether any modifications of the existing seepage collection and discharge system are warranted.

3.5 Coyote Dam

The final design of Coyote Dam is unprecedented and represents perhaps the first dam in the world to be specifically designed to cross an active fault and to have design measures to specifically accommodate fault offset of the foundation. The design called for symmetrical upstream and downstream slopes of 4:1 below el. 718 ft., 3.5:1 from el. 718 ft. to el. 758 ft., and 3:1 above el. 758 ft. The most important aspect of the adopted design was the unprecedented width of the crest of about 100 feet and the resulting thickness of the impervious core. The core consists of a homogeneous, well compacted, moderately plastic, sandy clay and constitutes the principal embankment material. This impervious core was designed symmetrically about the dam centerline, with a top width of 40 feet and an upstream and downstream slope of 2:1, thus resulting in a bottom width at the maximum section of 580 feet. Adjacent to the core on both the upstream and downstream sides, a free draining zone of clean gravel was placed at a slope of about 2.5:1. The gravel section is about 10 feet wide at the top near the crest and widens to in excess of 70 feet near the foundation. Heavy riprap was then placed to protect the gravels at the relatively flat slope inclinations quoted above. The extreme width of the core was specifically selected to accommodate in excess of 20 feet of horizontal offset of the Calaveras fault that had been predicted by Professor C. F. Tolman, the eminent Stanford geology professor, and Dr. G. D. Louderback, a member of the Consulting Board who also had served on the State Earthquake Investigation Commission following the catastrophic 1906 San Francisco earthquake. The overall intent of the unique design was to perform as follows: in the event an earthquake on the Calaveras fault created longitudinal or transverse cracking of the core, the design intended that the gravel would act as a crack stopper (i.e., it would fill the crack with gravel and significantly retard the velocity of flow) and the riprap would provide sufficient driving weight to force the gravel into any cracks. Although the earthquake-resistant design has a certain conceptual elegance, to our knowledge it has not been tested by co-seismic fault offset.

To reduce seepage beneath the dam, the upstream toe area of the core was seated in a cutoff trench which averaged 40 feet in depth across the main creek channel. The cutoff trench is curvilinear in plan (concave upstream) and located about 150 feet upstream of the centerline, about 80 feet downstream of the upstream core-foundation contact, at maximum section. The cutoff trench extended up the left abutment in order to fully expose the main trace of the Calaveras fault and to determine whether special treatment of the fault would be required.

During a 1977 safety investigation, three pneumatic piezometers were installed within the core and foundation areas. Two of these piezometers are still functioning and the results have provided us with some insight as to the piezometric response of the core. The new preliminary design has considered a series of piezometer locations that should provide a more reliable indication of the piezometric distribution within both the impervious core and the foundation, on both sides of the actively creeping Calaveras fault.

The preliminary design has considered the installation of seventeen (17) new piezometers at the dam; eleven (11) within the impervious core and six (6) within the foundation. A plan and cross section of Coyote Dam showing the planned locations and relative locations of the new instruments is presented in Figures H-10 and H-11, respectively. The instrument locations are summarized in Table H-5. A new transverse instrumentation section will be located at STA 3+75 near the maximum dam section. The new piezometers planned for the impervious core will be installed from three different drill hole locations. The presence of the gravel and riprap covering both the upstream and downstream shells creates difficulty in placing the instruments any further upstream or downstream than shown without a major materials handling problem. As shown in Figure H-11, new piezometers will be installed within the foundation in an attempt to determine if there is any preferential flow within the foundation due to the presence of the Calaveras and the "Right Abutment" fault. Several piezometers will also be installed within the impervious core at locations other than the maximum dam section to determine the piezometric distribution in other areas of the dam. In total, it is estimated that 12 drill holes will be required to install the 17 new piezometers.

In addition to the new piezometers discussed above, the preliminary design will include installation of one or more automated inclinometers and one strong motion accelerometer to act as a trigger to collect more data in the event of earthquake shaking. Details regarding these instruments will be developed during final instrumentation design.

Coyote Dam was outfitted with a new seepage collection system during the 1990/91 construction of the new outlet tunnel. Prior to this time, backwater from spillway discharges would collect and inundate the outlet discharge and the downstream toe area. The area was regraded and a modern seepage collection system and monitoring weir was constructed as a part of the overall project. The facilities appear to working properly and no additional work is considered necessary at this time.

3.6 Guadalupe Dam

Guadalupe Dam was designed and constructed as a zoned earthfill dam with an impervious zone in the upstream half and a pervious zone in the downstream half. Zoning within the dam was achieved by using relatively finer-grained materials in the impervious zone and coarser-grained materials in the pervious zone. Since the zoning was achieved by selective routing of construction hauling equipment within a single borrow area for the embankment fill there does not appear to be a significant distinction between the soil properties of the impervious zone and pervious zone. A cut-off trench was constructed slightly upstream of the midpoint of the "impervious" zone.

Foundation earth materials beneath the dam consist of sheared and closely fractured shales and sandstones of the Jurassic-Cretaceous Franciscan Complex. Clay soil up to 20 feet thick was encountered at the base of the left abutment and clay and gravel of an alluvial terrace was encountered in the channel area and beneath the lower right abutment. The soil materials described above encountered within the dam foundation were removed and used as suitable embankment fills, as appropriate.

Only one open well piezometer is still functioning at Guadalupe Dam. The new instrumentation design will provide monitoring of piezometric conditions throughout the embankment and foundation. The proposed locations and elevations of the nineteen (19) new piezometers for Guadalupe Dam are presented in plan and section in Figures H-12 and H-13 and summarized on Table H-6, respectively. As shown in Figure H-13, piezometers will be installed along two transverse sections; Section A-A' at STA 4+10 and Section C-C' at STA 2+50. As shown in Section B-B', which depicts a section taken through the U/S edge of the crest, one additional piezometer will be placed in each abutment. In total, it is estimated that 12 drill holes will be required to install the 19 new piezometers.

In addition to the new piezometers discussed above, the preliminary design has also considered the installation of one or more automated inclinometers and one strong motion accelerometer to act as a trigger to collect more data in the event of earthquake shaking. Details regarding these instruments will be developed during final instrumentation design.

To our knowledge, no seepage issues have ever been raised at Guadalupe Dam. The new instrumentation is designed to provide piezometric information in the downstream toe area. Based on the results of this future monitoring, an assessment will be made regarding whether any considerations should be given to the installation of a seepage collection and discharge system is warranted.

3.7 Rinconada Treated Water Reservoir Dam

The Rinconada Treated Water Reservoir Dam, which was constructed in 1969, is the smallest of the District dams under DSOD jurisdiction, at least with regard to volume of stored water. The embankment dam was built along one edge of a borrow pit, which has been modified to create a covered, lined-earth reservoir to store treated water from the adjacent Rinconada Water Treatment Plant. The 15-million gallon reservoir provides treated water storage and accommodates water demand fluctuations from the retail water customers.

The treatment plant and treated water reservoir were constructed on a foundation consisting of the Plio-Pleistocene Santa Clara Formation. In the vicinity of the reservoir the Santa Clara Formation consists of massive, well consolidated but poorly indurated siltstone and claystone and relatively impermeable weathered gravel in a matrix of silty sand and sandy clayey silt or sandy silty clay. The reservoir is contained within a modified borrow pit. During exploration, loose surficial materials were encountered in a colluvial filled swale in the vicinity of the treated water reservoir. During construction of the embankment the loose soils in the swale were over-excavated to a competent foundation, which was then backfilled with compacted Santa Clara Formation materials, consisting of silty sand with varying amounts of clay and gravel. The over-excavated swale area represents the maximum section of the embankment. The majority of the reservoir is founded in cut.

The treated water reservoir is entirely covered by a major roof section. Although not shown on any drawing presented herein, the roof, which is supported by 16-in reinforced concrete columns, is shaped with its main ridge line running along the long axis of the reservoir (i.e., essentially north-south). Therefore, rather than being controlled by rainfall and discharge from a watershed, water levels in the reservoir are governed by treated water production and conveyance to and from the treatment plant. Consequently, fluctuations in piezometric levels near the reservoir reflect changes in groundwater conditions due predominantly to surface water infiltration (rainfall) and not seepage through the embankment dam. Since the treated water reservoir is lined, piezometric levels should not be directly affected by changes in reservoir water levels unless the liner develops a leak. Such a leak would hopefully be detected as increased seepage through the underdrain system. The seepage discharge data collected at the site reflects the total discharge measured from four areas within the underdrain collector pipe system. Between September 1992 and March 1993, the deteriorating original reservoir liner (butyl rubber) was replaced with a geotextile fabric and hypalon liner. Prior to initiating the liner replacement project, total underdrain discharge seeping through the butyl rubber liner was as high as 11.5 gallons per minute (gpm). Since the new liner was installed, seepage discharge has tapered off to 0 gpm and open well piezometers have stabilized; this indicates that the hypalon liner replacement is performing well and that there is no detectable leakage through the liner

The proposed locations and elevations of the nine (9) new piezometers for the Rinconada TWR Dam are presented in plan and section in Figures H-14 and H-15 and summarized on Table H-7, respectively. As shown in Figure H-14, and based on our review of the history of previous instrumentation results, the area of embankment fill is rather limited and just one section (Section A-A') has been selected for instrumentation through the maximum embankment section. The primary objectives of the piezometers are: 1) to determine the piezometric conditions acting within the embankment and foundation; and 2) to provide piezometric information in the event any leaks may develop within the hypalon liner.

3.8 Stevens Creek Dam

Stevens Creek Dam is the only District dam that has required a seismic retrofit. Following a detailed dynamic analysis in 1985, it was determined that there was a potential for significant displacement to occur within the dam if a maximum possible earthquake of M=8.3 were to occur on the nearby San Andreas fault. As a result, major buttresses were placed over the upstream and downstream shells to provide added earthquake resistance, and the dam crest was raised about 10 feet. The new dam has about 19.2 feet of freeboard. When the new buttresses were constructed in 1985, 9 pneumatic piezometers were installed and all are still operational and functioning. There is no plan to discontinue the use of these instruments. They will continue to be read monthly, at least until the new instruments have been installed and can be used to verify the pore pressure response throughout the dam.

Thirteen (13) new piezometers are planned for the preliminary design and the locations are presented in Figures H-16 and H-17 and summarized on Table H-8. As shown in Figure H-16, there will be one new instrumentation section located at STA 8+50, where nine (9) new piezometers are planned. It will be necessary to remove a portion of the upstream concrete facing to install piezometers 72-7-1 through 72-7-4. Two separate drill holes will be required. These piezometers will augment the existing pneumatic piezometers plus provide the added benefit of being automated with daily readings to determine different piezometric responses depending on reservoir fluctuations. Six (6) new piezometers are also planned to be installed from the crest of the original dam. As shown in Figure H-17, additional piezometers will be installed within the foundation and central "pervious" zone in the left, central and right abutment sides of the dam.

The locations of several of the piezometers have been specifically considered in order to develop information on the performance of the foundation materials as the reservoir level fluctuates at different times through the water year. The foundation earth materials beneath the dam consist of alluvial terrace deposits (alluvium) underlain by rocks of the Santa Clara Formation. The alluvium is present over only a portion of the foundation. Although not shown on the "As-Built" drawings, it is likely that the alluvium was left in-place in the main channel section of the former creek channel, but was not left in-place in the abutments. According to the geologic investigation performed for the seismic retrofit (Wahler, 1982), field geologists described the alluvium as ranging from loose to medium dense sand, with less than 15 percent fines, to soft/firm clayey sand with gravel. The alluvium ranges from about 10 to 15 feet thick and is present beneath the central and right abutment areas of Stevens Creek Dam. Piezometers 72-7-2, -5, and -8 are intended to be placed at the approximate center of the alluvium (if it is present at STA 8+50) in order to study the piezometric response of these materials. The installation of the piezometers will also provide the opportunity to sample and subsequently test these materials. In total, it is estimated that 11 drill holes will be required to install the 13 new piezometers.

In addition to the new piezometers discussed above, the preliminary design has also considered the installation of one or more automated inclinometers and one strong motion accelerometer to act as a trigger to collect more data in the event of earthquake shaking. Details regarding these instruments will be developed during final instrumentation design.

A new seepage collection and monitoring system was installed at the downstream toe of the downstream buttress during the seismic retrofit work of 1985. The existing pneumatic piezometers indicate that the drainage system is working fine. We will continue to monitor the new instruments in order to evaluate the effectiveness of the existing system.

3.9 Uvas Dam

3.9.1 Piezometer Requirements

Since its construction in 1956 and 1957 by the Gavilan Water District, Uvas Dam has had recurring seepage problems. Several attempts have been made to mitigate and control seepage flow through the embankment, foundation, and abutments. These efforts have included installation of several drains, grout curtains, and 6-inch diameter relief wells (RW-1 through RW-4). The relief wells are now used for monitoring piezometric levels.

The dam was constructed on Jurassic-Cretaceous Franciscan Complex rocks. The rock types near the dam include basalt, greenstone, limestone, chert, shale, and sandstone. The bedrock underneath the dam is blanketed with Quaternary deposits, which have been interpreted as consisting of artificial fill, colluvium, channel, terrace, and landslide deposits. During construction of the dam, a grout curtain was installed along the centerline of the cutoff trench. In May 1958, a second grout curtain, consisting of 18 grout holes, was placed between the spillway weir and the left end of the dam. In 1996, the District installed a double-row grout curtain in the foundation bedrock of the right abutment. This later effort appears to have significantly reduced the seepage, especially that emanating through the right abutment.

Uvas Dam was constructed as a zoned earth dam using Franciscan-derived material as embankment fill. As-built drawings show upstream and downstream "less pervious" zones and a central core that extends partly upstream identified as "select impervious". An inclined chimney drain at the downstream face of the core and a horizontal blanket drain along the downstream toe of the dam were also incorporated into the design. Previous investigations found that the embankment materials are generally gravelly, and range from weathered bedrock fragments to gravels to clayey sands and gravels. After construction, several drains were installed within the embankment to reduce pore pressures. These included horizontal drains near the left abutment, a downstream toe drain, several 6-inch diameter monitoring wells (also referred to as "relief wells") along the downstream berm, and a chimney drain at the downstream face of the core and interceptor drain along the downstream toe.

Since its original construction, thirty six (36) open well piezometers have been installed at the dam. These piezometers will continue to be read on a monthly basis until the new vibrating gauge piezometers and automated recording system has been installed and hooked up and a reasonable history developed for comparison.

The proposed locations and elevations of the twenty four (24) new piezometers for Uvas Dam are presented in plan and section in Figures H-18 and H-19 and summarized on Table H-9, respectively. As shown in Figure H-18, and based on our review of the history of previous instrumentation results, piezometers will be installed along one major transverse cross section A-A' (STA 8+50) and two longitudinal sections that include Section B-B' along the dam crest and Section C-C' along the downstream bench (elev. 433± ft). Both B-B' and C-C' extend from the left abutment to the right abutment. The presence of the inclined drain limits the ability to install instruments along a section of the downstream embankment below the drain using rotary wash drilling methods due to concerns that the mud would contaminate the inclined drain. Any piezometers in this area of the drain will require the use of hollow stem auger with special consideration given to grouting the hole. According to the preliminary design, the new instruments will measure piezometric pressures within both the embankment and underlying landslide and alluvial terrace materials and bedrock.

It is estimated that 21 drill holes will be required to install the 24 new piezometers planned for Uvas Dam. The holes will total about 1,325 feet through the embankment and 185 feet into underlying bedrock or foundation soils.

3.9.2 Weirs and Drains – General Description and History

At Uvas Dam the seepage is measured at four weirs; three are located along the base of the lower downstream berm and one is located near the downstream toe. All four weirs are metal plate V-notch weirs that discharge from concrete boxes. Formerly, some seepage was occasionally measured at temporary drain pipes installed in four shallow trenches that were cut into the downstream face (normal to the axis) upslope of the berm. At the request of DSOD, these trenches have been backfilled and marked in the field with posts (as of March 1999). Based on the lack of observed excess seepage at the ground surface downstream of the dam, it appears that the weirs capture a large component of seepage from the dam and its foundation. Weir 1 provides seepage information from the left abutment foundation, Weir 2 provides seepage information from the embankment, and Weir 3 provides seepage information from the right abutment foundation. The Toe Drain Weir provides seepage from the toe drain. It should be noted however that the toe drain weir has not provided reliable discharge readings since about December 1997 because it has been submerged beneath high water in an adjacent pool. The high water is due to the accumulation of a gravel deposit that blocks the far (downstream) end of the pool.

Based on a review of previous maintenance and construction records, it appears that when the downstream inclined drain and buttress was installed that the blanket drain was not extended out to the toe of the buttress. It is not clear how this shortcoming may be influencing the overall seepage pattern or volume.

3.9.3 New Seepage Collection System

In order to deal with the seepage issues described above, a new seepage collection system is planned for the downstream toe area. It appears that the seepage collection system that collects abutment seepage that discharges into the three weirs located on the lower berm is functioning properly. The functionality of seepage collection and weir discharge along the toe of the dam is not as clear. We propose to reconstruct the toe drain seepage collection and weirs to current standards. The construction associated with this work will include excavating the downstream toe area and exposing firm underlying bedrock, colluvium, and downstream shell materials, and then installing necessary drain pipes, drain materials, and appropriate geotextile fabric to adequately collect and discharge the collected seepage. The District has not yet evaluated the requirements for a soil/bedrock exploration program in order to develop the necessary design parameters, dimensions, and lateral extent of the required seepage protection facilities. It should also be noted that property rights-of-way appear to be an issue along the downstream toe. It may be necessary to replace the existing soil section in the downstream toe area with a more stable rock fill zone that can be completed at a slightly steeper inclination. The necessary design for the seepage collection system will, most likely, be completed and submitted to DSOD with the preliminary plans and specifications for the instrumentation application.

In addition to the new piezometers discussed above, the preliminary design has also considered the installation of one or more automated inclinometers and one strong motion accelerometer to act as a trigger to collect more data in the event of earthquake shaking. Details regarding these instruments will be developed during final instrumentation design.

3.10 Vasona Dam

Vasona Dam, one of the Districts original six dams built in 1935, has been significantly altered since its construction. Prior to the construction of Austrian Dam (owned and operated by the San Jose Water Company) and Lexington (now Lenihan) Dam in the mid-1950's, Vasona Dam provided the major flood control on Los Gatos Creek. Since the construction of both Austrian Dam and Lenihan Dam, a significant amount of the rainfall falling within the Los Gatos Creek watershed is captured and stored behind these two major dams. As a result, the significant flood capacity in the lower Los Gatos Creek area that was originally provided by Vasona Dam is no longer needed. As a result, the majority of the smaller (upper) radial gates at the dam were permanently removed in 1971. The removal of these gates (which were located on the west side of the larger lower radial gates) caused a reduction in lake capacity from 750 acre-ft to 400 acre-ft, and permanent lowering of the maximum reservoir elevation of Vasona Lake from 302.6 feet to 297.6 feet (NAVD 1988). The larger lower radial gates are still used for flood release if needed. The primary use of Vasona Dam today is to provide control of water released to the downstream seepage recharge ponds operated by the District and for recreation at Vasona Reservoir.

A safety investigation of Vasona Dam was completed in 1977 (Wahler, 1977). Five (5) open well piezometers were installed during the investigation and are still operational. The proposed locations and elevations of the eight (8) new piezometers for Vasona Dam are presented on a plan and sections in Figures H-20 and H-21 and summarized on Table H-10, respectively. Based on our review of the history of previous instrumentation results, two sections have been selected for instrumentation; STA 1+80 on the right (east) side of the spillway and STA 8+00 on the left (west) side of the spillway. As shown in Figure H-21, four (4) piezometers will be installed in two drill holes along each section, including two beneath the centerline of the embankment and two beneath the mid-height of the downstream shell. Since the reservoir remains essentially constant, automating the instruments does not appear to be warranted. For the preliminary design, we have assumed that the piezometers would be hooked up to a data logger that would be programmed to obtain piezometer readings on a daily basis and that the data would be downloaded once per month.

The relatively pervious foundation at Vasona Dam acts as an underdrain. To our knowledge, no seepage issues have ever been raised at the dam. The new instrumentation is intended to provide additional information regarding the piezometric conditions within the downstream shell and toe area. Based on the results of this future monitoring, an assessment will be made regarding whether a seepage collection and discharge system is warranted.

3.11 Other District Facilities

There is at least one other facility that may require future instrumentation and become a part of the District Dam Safety Program. The Coyote Valley Research Project (CVRP) is a planned new industrial development adjacent to Fisher Creek in southeastern San Jose. Due to its potential impact on Fisher Creek, the developer has proposed a major realignment and flood diversion structure as part of the development. A review of the planned facilities indicates that the proposed facilities come under DSOD jurisdiction. DSOD has reviewed the design and construction of the planned creek realignment and related flood diversion structures. The development-related construction work, which was originally scheduled for 2002, has been delayed due to the economic downturn. The District has completed its review of the preliminary design and requested that several survey monuments be installed along the crest of several structures. When the planned CVRP structures are completed and accepted by DSOD and the District, they will be incorporated into the District's Dam Safety Program, which will include complete instrumentation monitoring and safety inspections.

4.0 References

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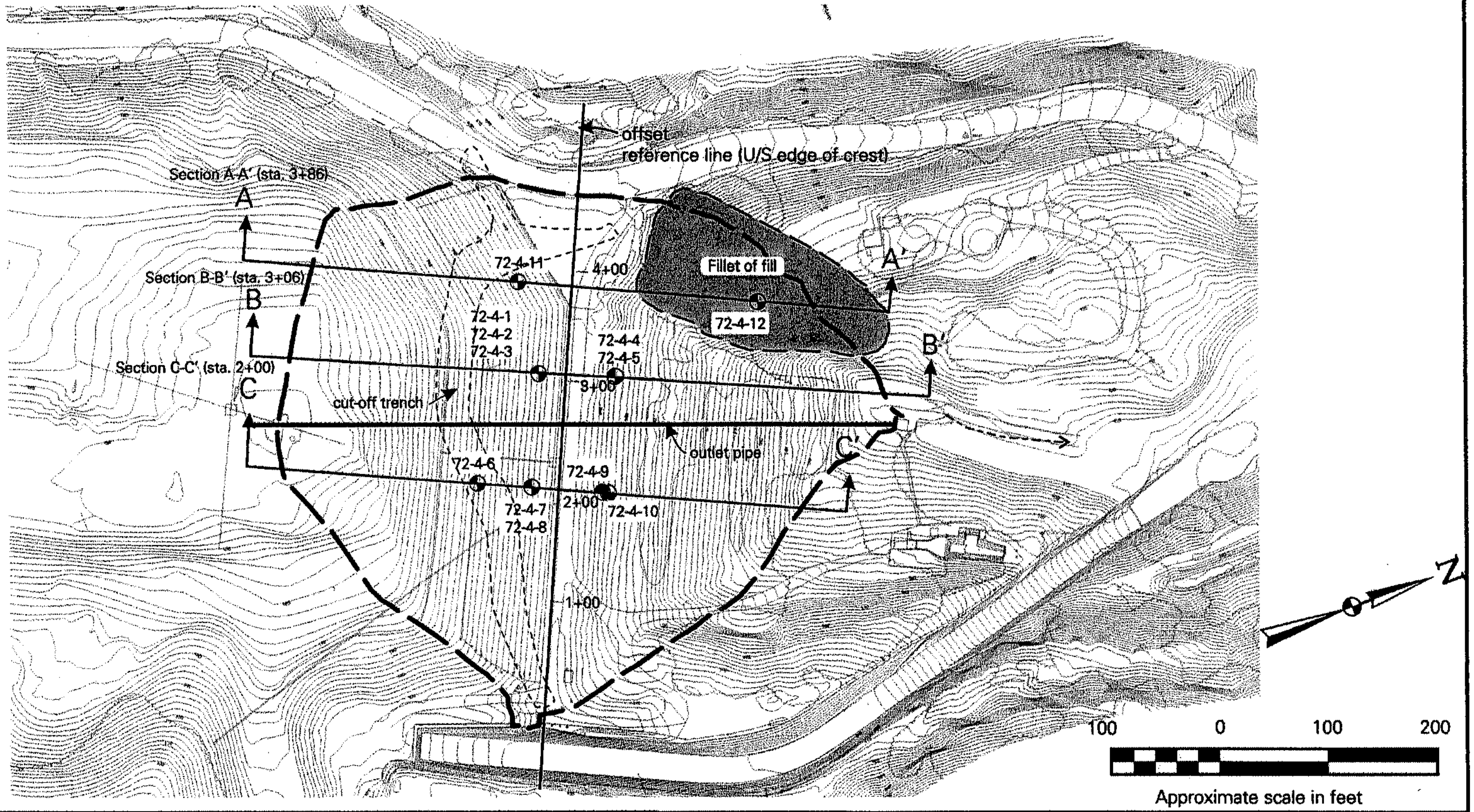


Notes:
1) File: W:\geology\dam safety program\dam safety rport\figures\H-2.cdr
2) basis of elevations is NAVD 1988-elevations are 2.03' higher than NGVD 1929

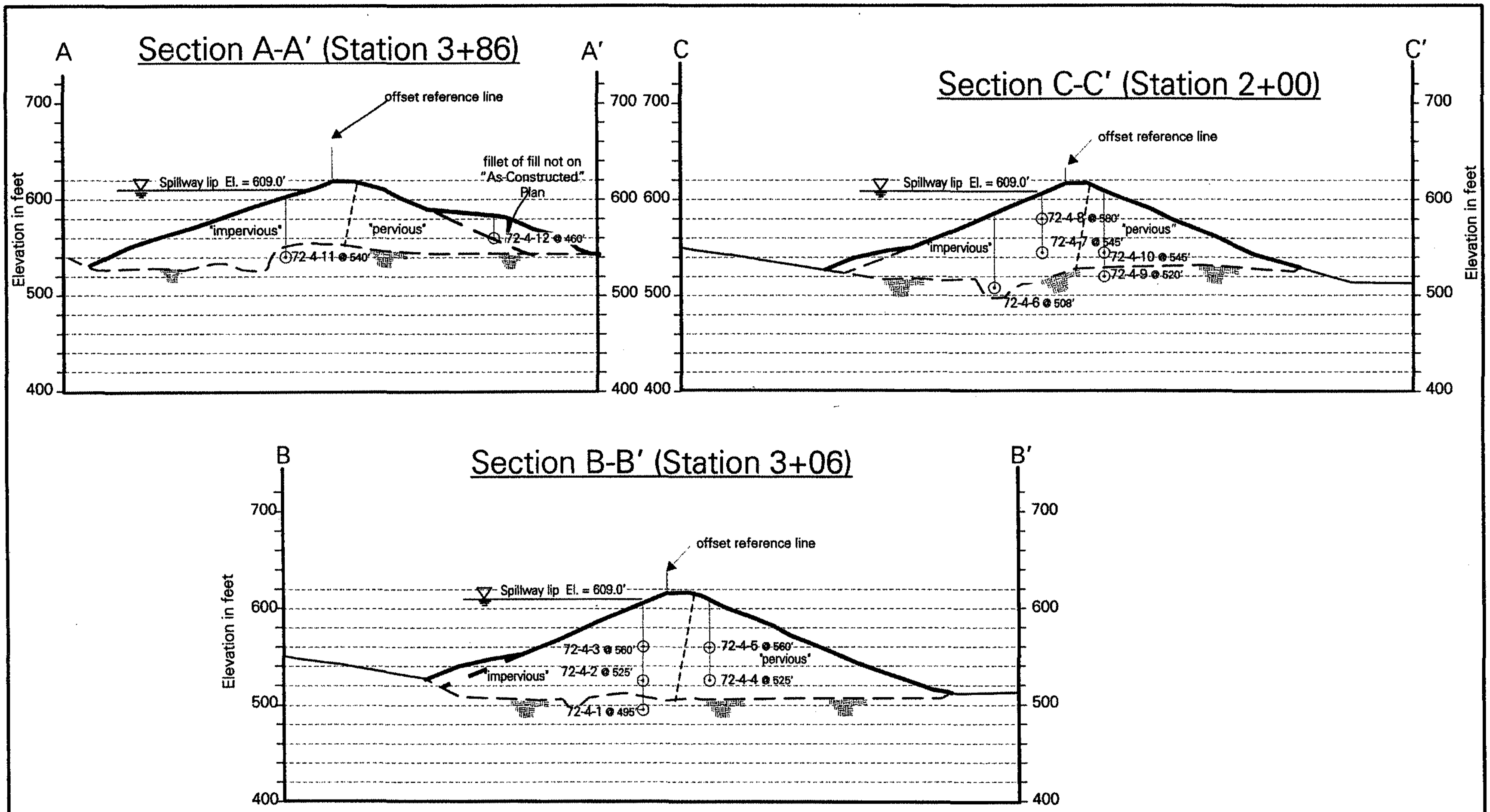
DATE 01/17/02
DESIGN RLV
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

New Piezometer Locations

Almaden Dam (No. 72-4)-Plan

SCALE 1"=100'
FIGURE H-2



Notes: 1) file W:\geology\dam safety program\dam safety rport\figures\H-3.cdr
2) elevations are based on NAVD 1988 - elevations on this section are 2.03 feet higher than NGVD 1929

DATE	01/17/02
DESIGN	RLV
DRAWN	JLN
CHECKED	RLV

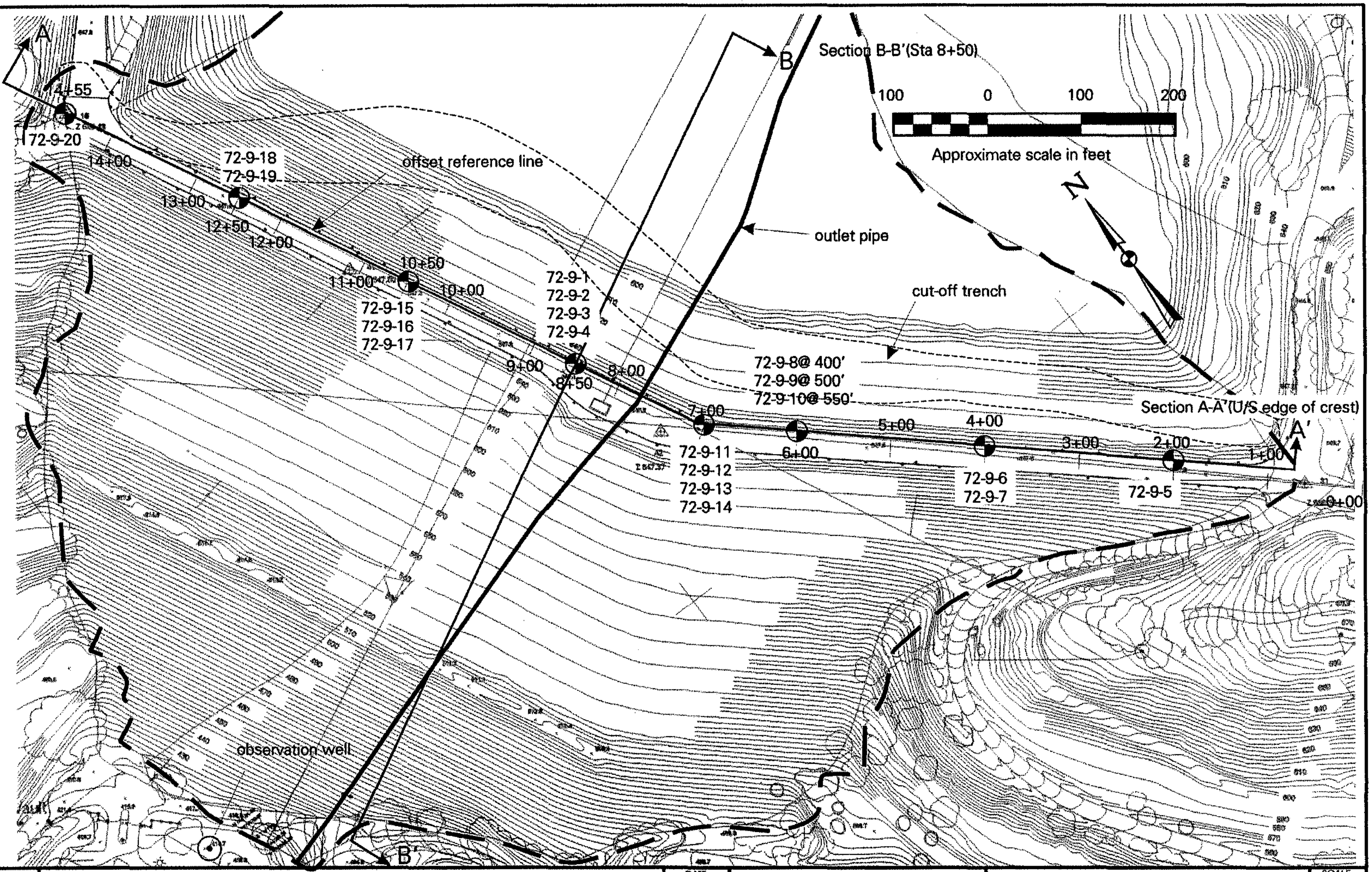
Santa Clara Valley Water District

New Piezometer Locations

Almaden Dam (No. 72-4)-Sections

SCALE
1" = 100

FIGURE
H-3



Notes:
1) File: W:\geology\dam safety program\dam safety rport\figures\H-4.cdr
2) basis of elevations is NAVD 1988-elevations are 2.80' higher than NGVD 1929

DATE	01/14/02
DESIGN	RLV
DRAWN	JLN
CHECKED	RLV

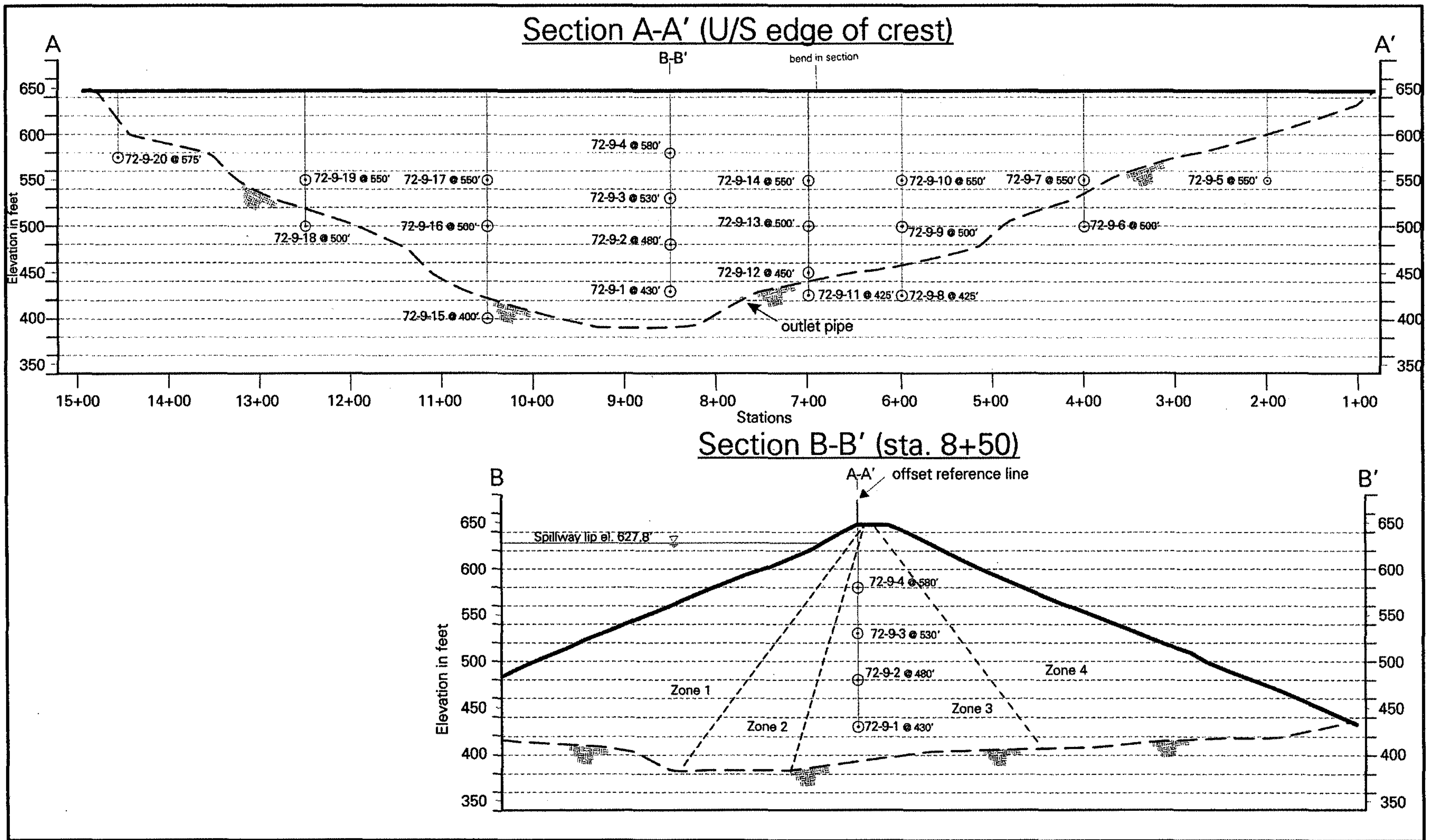
Santa Clara Valley Water District

New Piezometer Locations

Leroy Anderson Dam (No. 72-9)-Plan

SCALE
1"=100'

FIGURE
H-4



Section A-A' (U/S edge of crest)

Section B-B' (sta. 8+50)

Notes:
1) File: W:\geology\dam safety program\dam safety rport\figures\H-5.cdr
2) basis of elevations is NAVD 1988-elevations are 2.80' higher than NGVD 1929

DATE	01/14/02
DESIGN	RLV
DRAWN	JLN
CHECKED	RLV

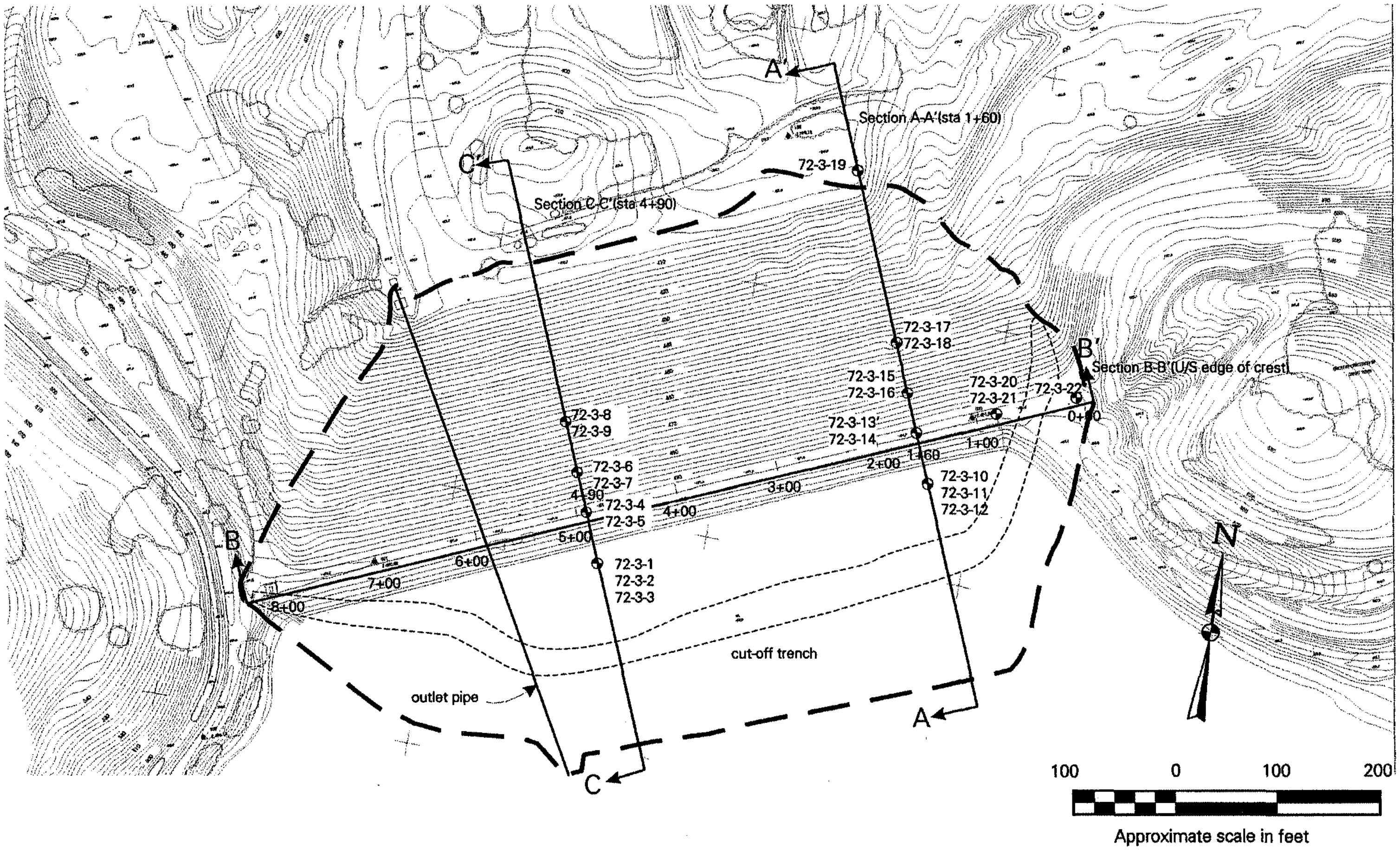
Santa Clara Valley Water District

New Piezometer Locations

Leroy Anderson Dam (No. 72-9)-Sections

SCALE 1"=100'

FIGURE H-5



Notes: 1) file W:\geology\dam safety program\dam safety rport\figures\H-6.cdr
2) elevations are based on NAVD 1988 - elevations on this section are 2.89 feet higher than NGVD 1929

DATE	01/03/02
DESIGN	RLV
DRAWN	JLN
CHECKED	RLV

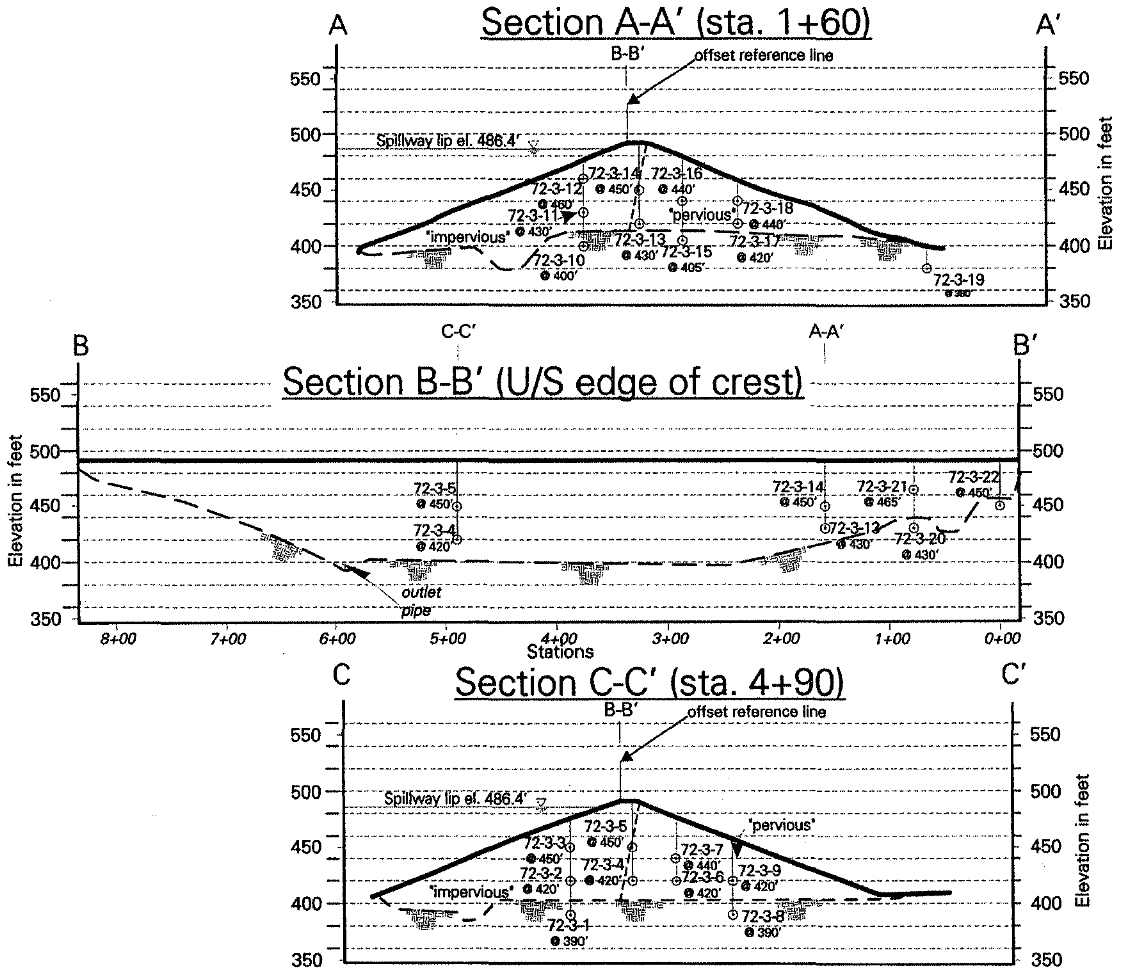
Santa Clara Valley Water District

New Piezometer Locations

Calero Main Dam (No. 72-3)-Plan

SCALE 1"=100'

FIGURE H-6



Notes: 1) file W:\geology\dam safety program\dam safety rport\figures\H-7.cdr
2) elevations are based on NAVD 1988 - elevations on this section are 2.89 feet higher than NGVD 1929

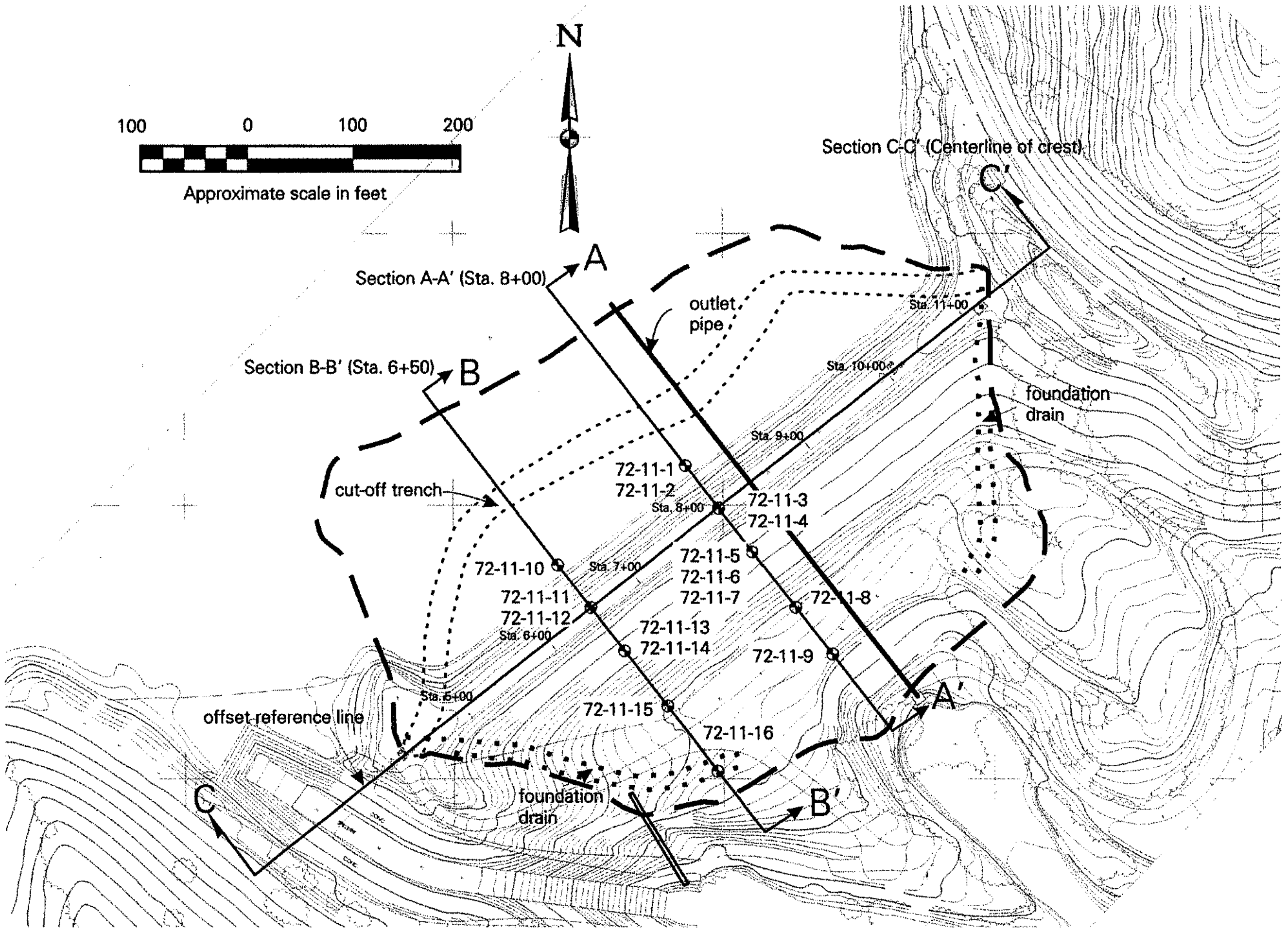
DATE	12/28/01
DESIGN	RLV
DRAWN	JLN
CHECKED	RLV

Santa Clara Valley Water District

New Piezometer Locations

Calero Main Dam (No. 72-3)-Sections

SCALE	1"=100'
FIGURE	H-7



Notes:
1) file W:\geolgy\dam safety program\dam safety rport\figures\H-8.cdr
2) basis of elevations is NAVD 1988-elevations are
3.19 feet higher than NGVD 1929

DATE	01/03/02
DESIGN	RLV
DRAWN	JLN
CHECKED	RLV

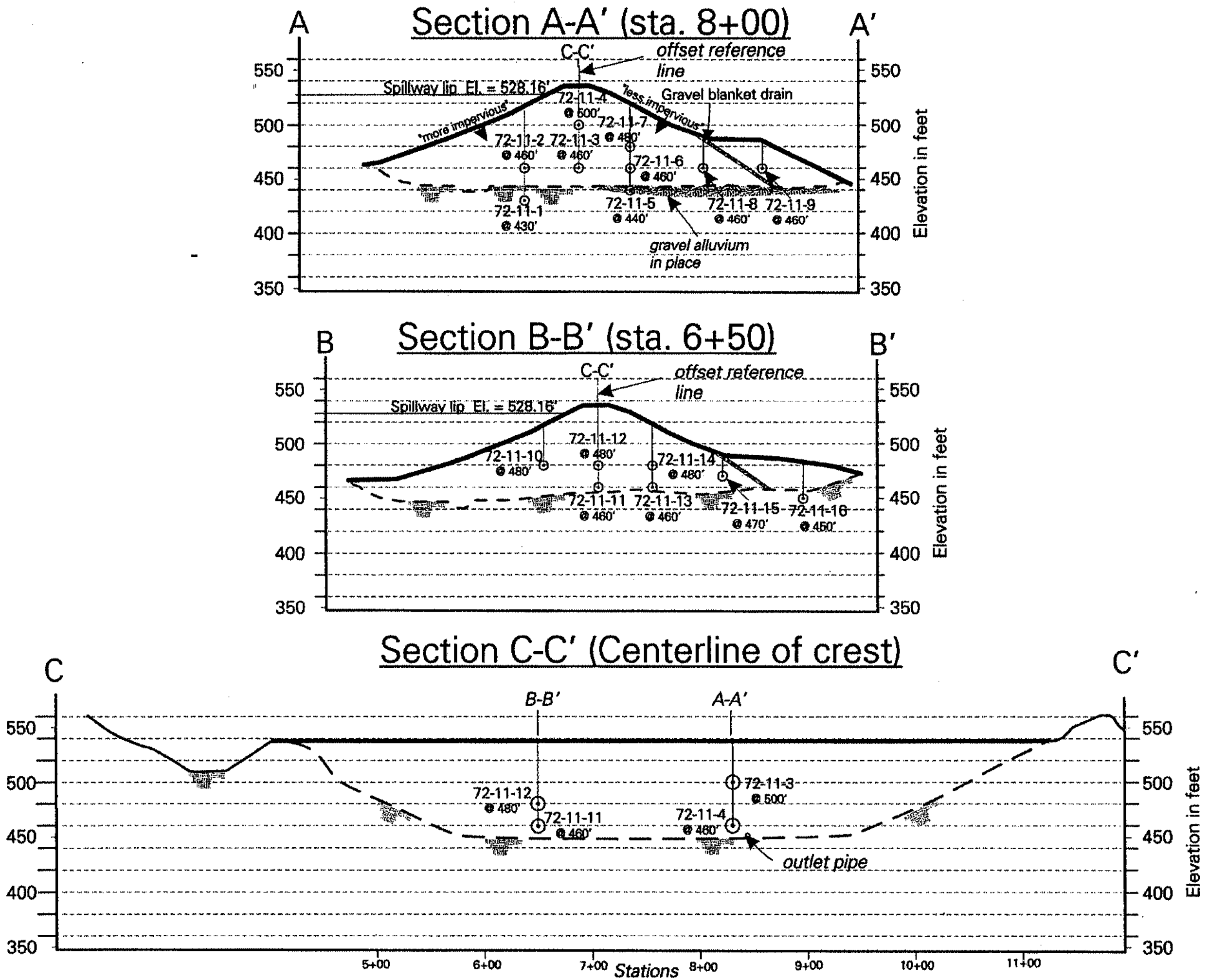
Santa Clara Valley Water District

New Piezometer Locations

Chesbro Dam (No. 72-11)-Plan

SCALE
1"=100'

FIGURE
H-8



Notes:
1) file W:\geology\dam safety program\dam safety rport\figures\H-9.cdr
2) basis of elevations is NAVD 1988-elevations are
3.19 feet higher than NGVD 1929

DATE
01/03/02
DESIGN
RLV
DRAWN
JLN
CHECKED
RLV

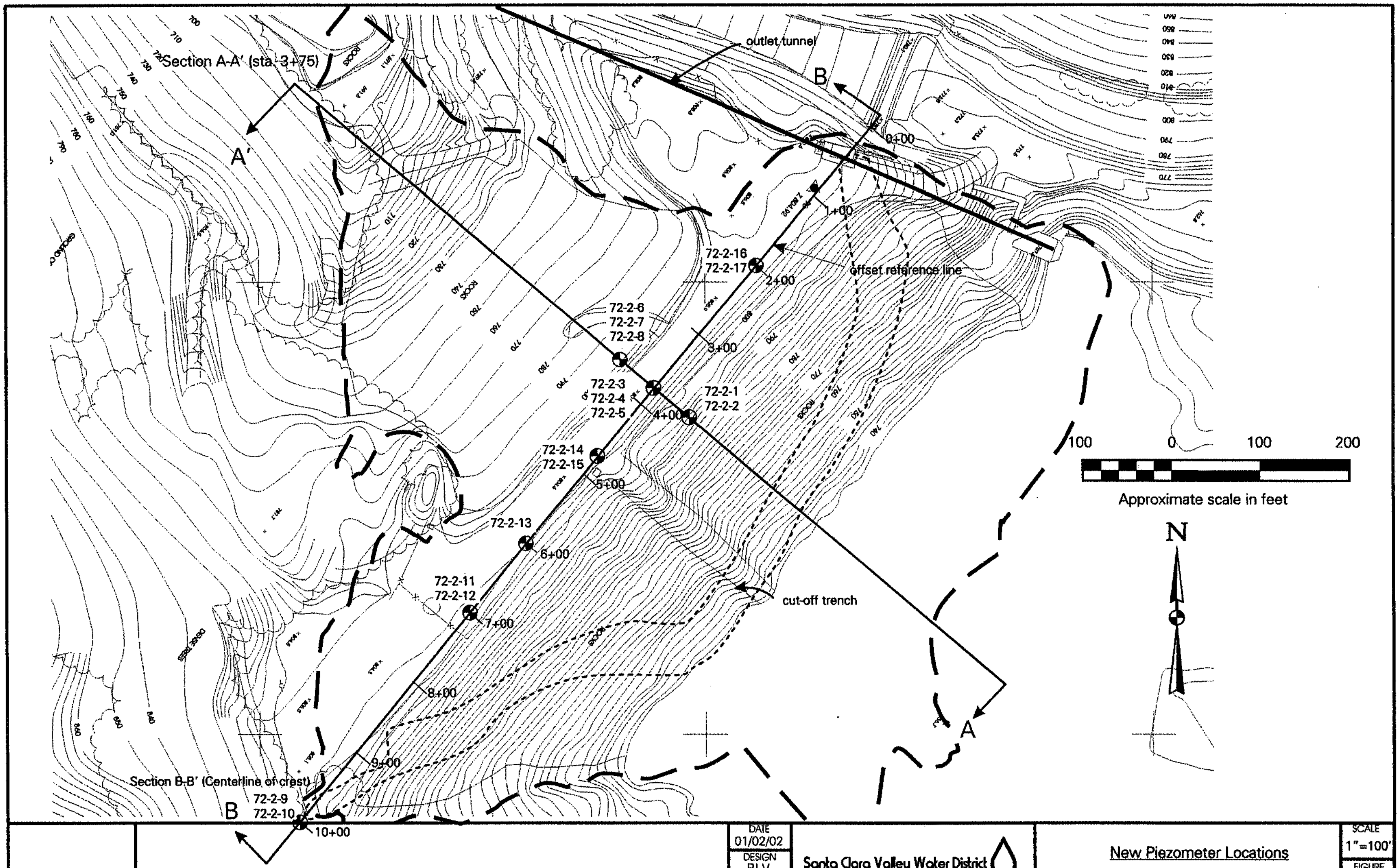
Santa Clara Valley Water District

New Piezometer Locations

Elmer J. Chesbro Dam (No. 72-11)-Sections

SCALE
1" = 100'

FIGURE
H-9



Notes: 1) file W:\geology\dam safety program\dam safety rport\figures\H-10.cdr
2) basis of elevations is NAVD 1988-elevations are 2.87' higher than NGVD 1929

DATE	01/02/02
DESIGN	RLV
DRAWN	JLN
CHECKED	RLV

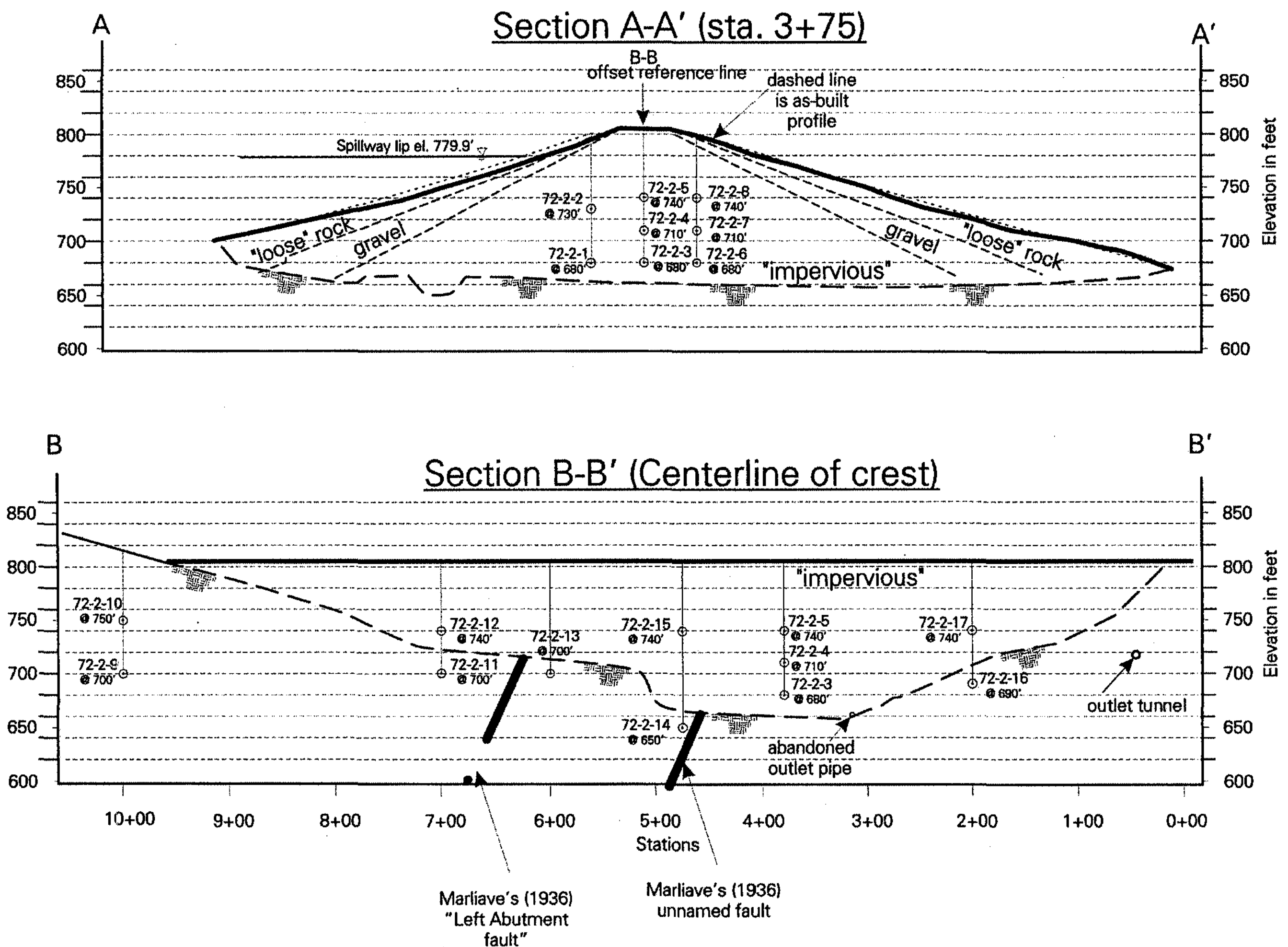
Santa Clara Valley Water District

New Piezometer Locations

Coyote Dam (No. 72-2)-Plan

SCALE
1"=100'

FIGURE
H-10



Notes: 1) file W:\geology\dam safety program\dam safety rport\figures\H-11.cdr
2) basis of elevations is NAVD 1988-elevations are 2.87' higher than NGVD 1929

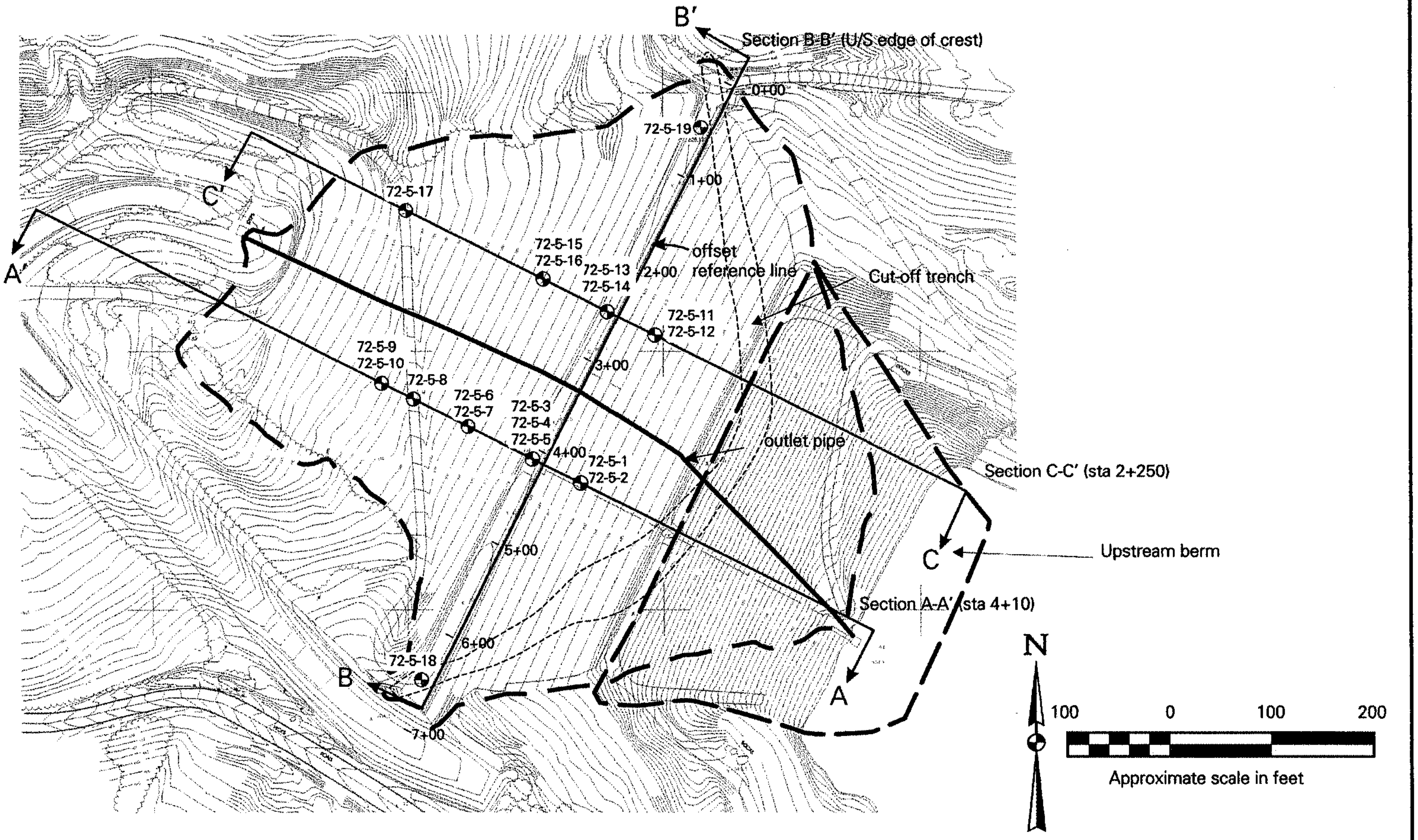
DATE
01/02/02
DESIGN
RLV
DRAWN
JLN
CHECKED
RLV

Santa Clara Valley Water District

New Piezometer Locations

Coyote Dam (No. 72-2)-Sections

SCALE
1"=100'
FIGURE
H-11



Notes:
1) File: W:\geology\dam safety program\dam safety rport\figures\H-12.cdr
2) basis of elevations is NAVD 1988-elevations are 2.28′ higher than NGVD 1929

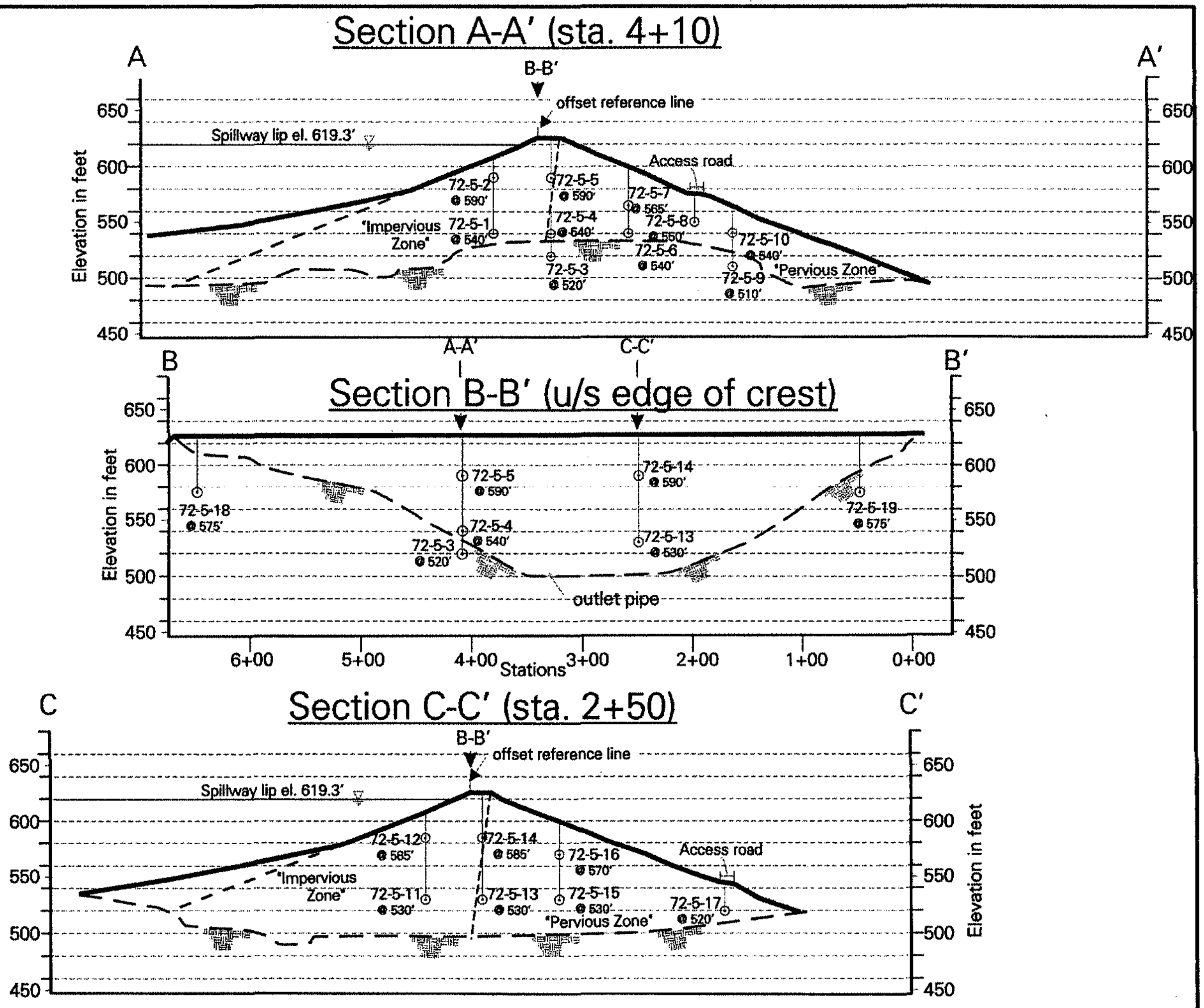
DATE
12/26/01
DESIGN
RLV
DRAWN
JLN
CHECKED
RLV

Santa Clara Valley Water District

New Piezometer Locations

Guadalupe Dam (No. 72-5)-Plan

SCALE
1″=100′
FIGURE
H-12



Section A-A′ (sta. 4+10)

Section B-B′ (u/s edge of crest)

Section C-C′ (sta. 2+50)

Notes:
1) File: W:\geology\dam safety program\dam safety rport\figures\H-13.cdr
2) basis of elevations is NAVD 1988-elevations are 2.28′ higher than NGVD 1929

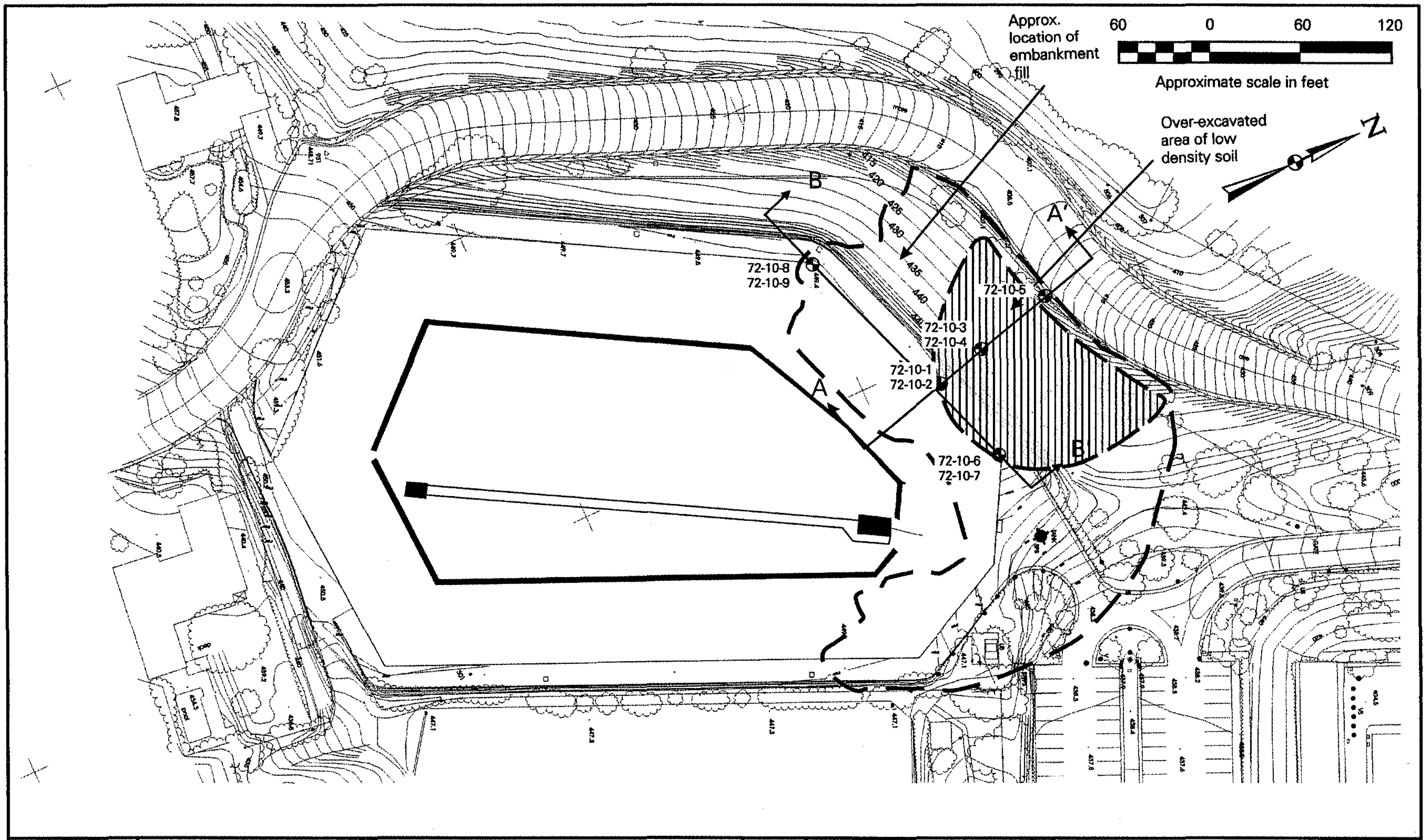
DATE
12/26/01
DESIGN
RLV
DRAWN
JLN
CHECKED
RLV

Santa Clara Valley Water District

New Piezometer Locations

Guadalupe Dam (No. 72-5)-Sections

SCALE
1″=100′
FIGURE
H-13


Approx. location of embankment fill
60
0
60
120
Approximate scale in feet
Over-excavated area of low density soil
N
B
A'
A
B'
415
420
425
430
435
440
72-10-8
72-10-9
72-10-5
72-10-3
72-10-4
72-10-1
72-10-2
72-10-6
72-10-7

Notes:
1) file W:\geology\dam safety program\dam safety rport\figures\H-14.cdr
2) basis of elevations is NGVD 1929

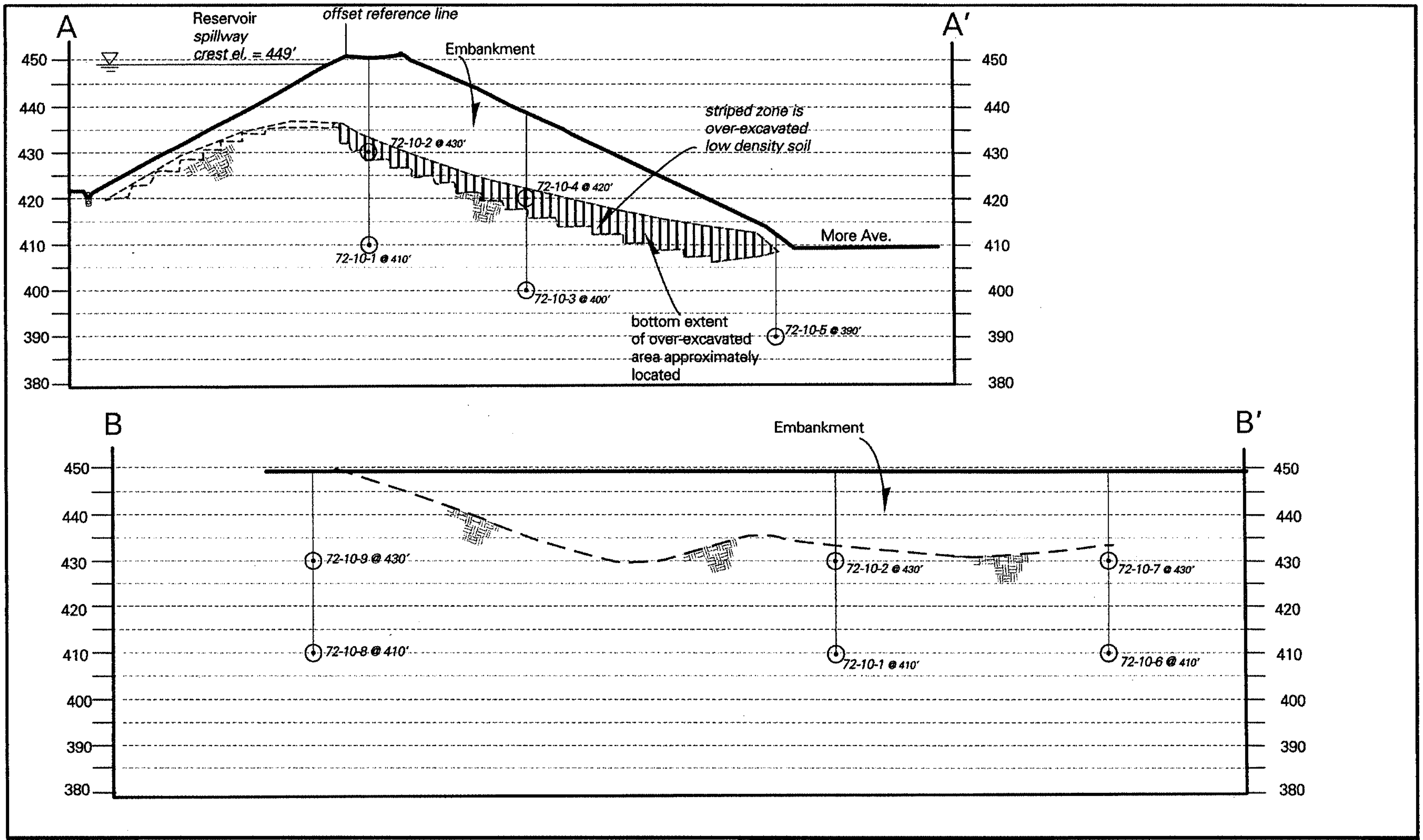
DATE 1/21/02
DESIGN RLV
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

New Piezometer Locations

Rinconada TWR Dam (No. 72-10)-Plan

SCALE 1" = 60'
FIGURE H-14



Notes:
1) file W:\geology\dam safety program\dam safety rport\figures\H-15.cdr
2) basis of elevations is NGVD 1929

DATE 1/21/02
DESIGN RLV
DRAWN JLN
CHECKED RLV

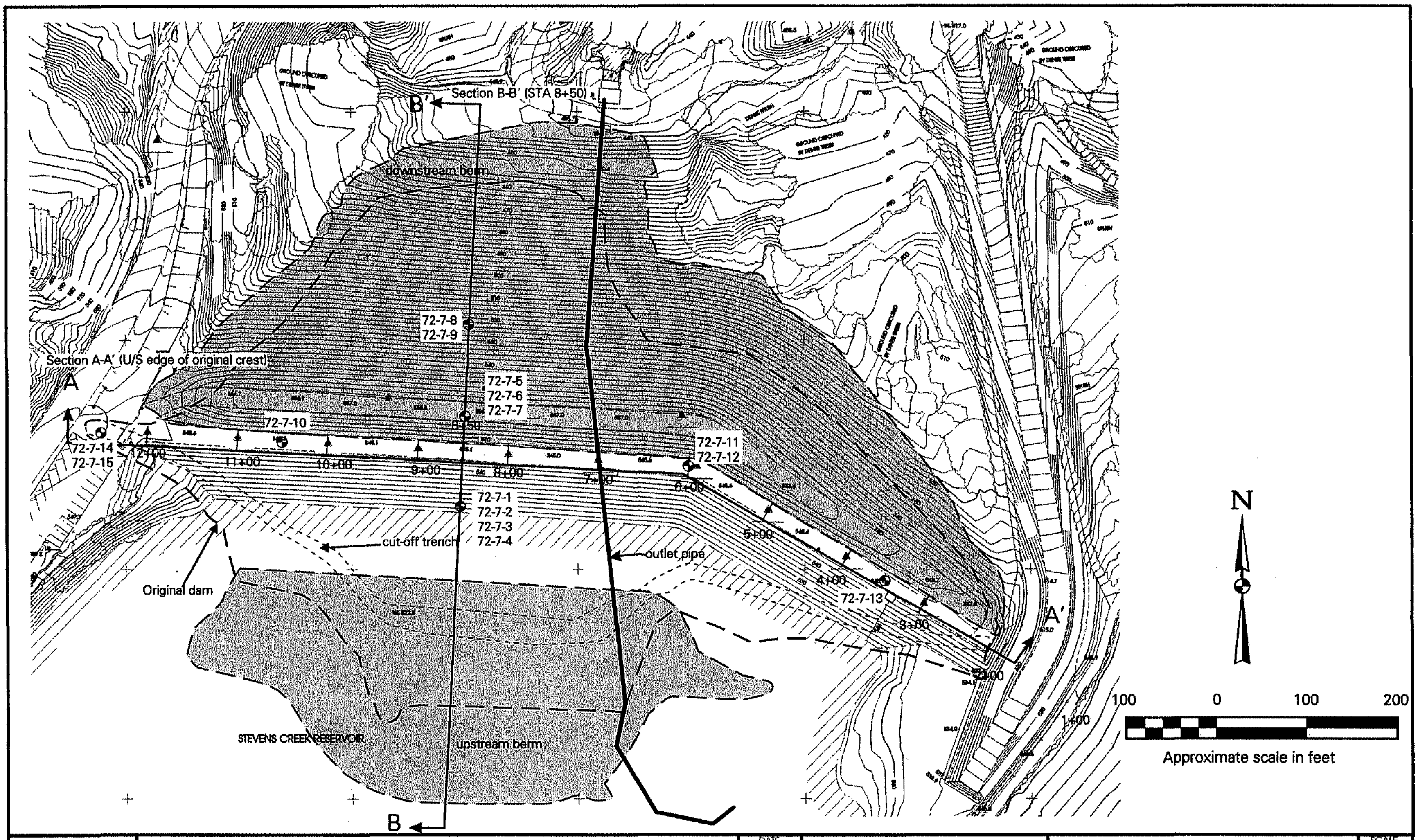
Santa Clara Valley Water District

New Piezometer Locations

Rinconada TWR Dam (No. 72-10)-Sections

SCALE 1" = 20′

FIGURE H-15



Notes:
1) File: W:\geology\dam safety program\dam safety rport\figures\H-16.cdr
2) basis of elevations is NAVD 1988-elevations are 0.7' higher than NGVD 1929

DATE
01/16/02
DESIGN
RLV
DRAWN
JLN
CHECKED
RLV

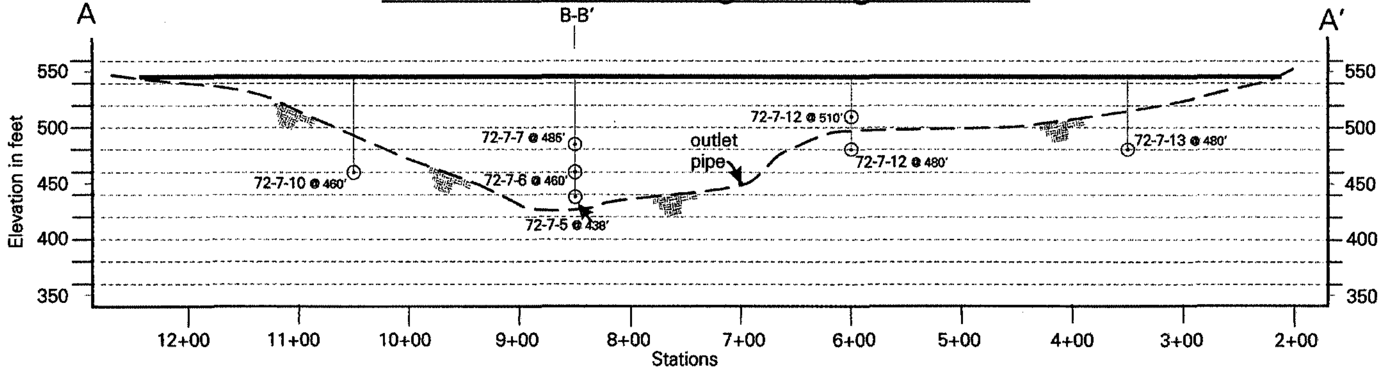
Santa Clara Valley Water District

New Piezometer Locations

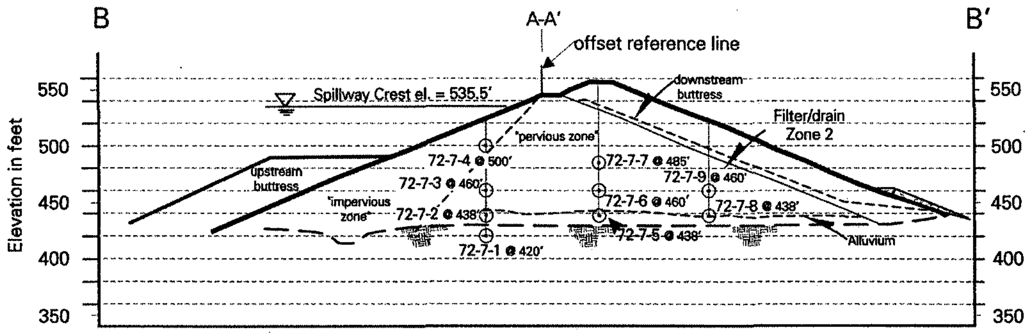
Stevens Creek Dam (No. 72-7)-Plan

SCALE
1"=100'
FIGURE
H-16

Section A-A′ (U/S edge of original crest)



Section B-B′ (sta. 8+50)



Notes:
1) File: W:\geology\dam safety program\dam safety rport\figures\H-17.cdr
2) basis of elevations is NAVD 1988-elevations are 0.7′ higher than NGVD 1929

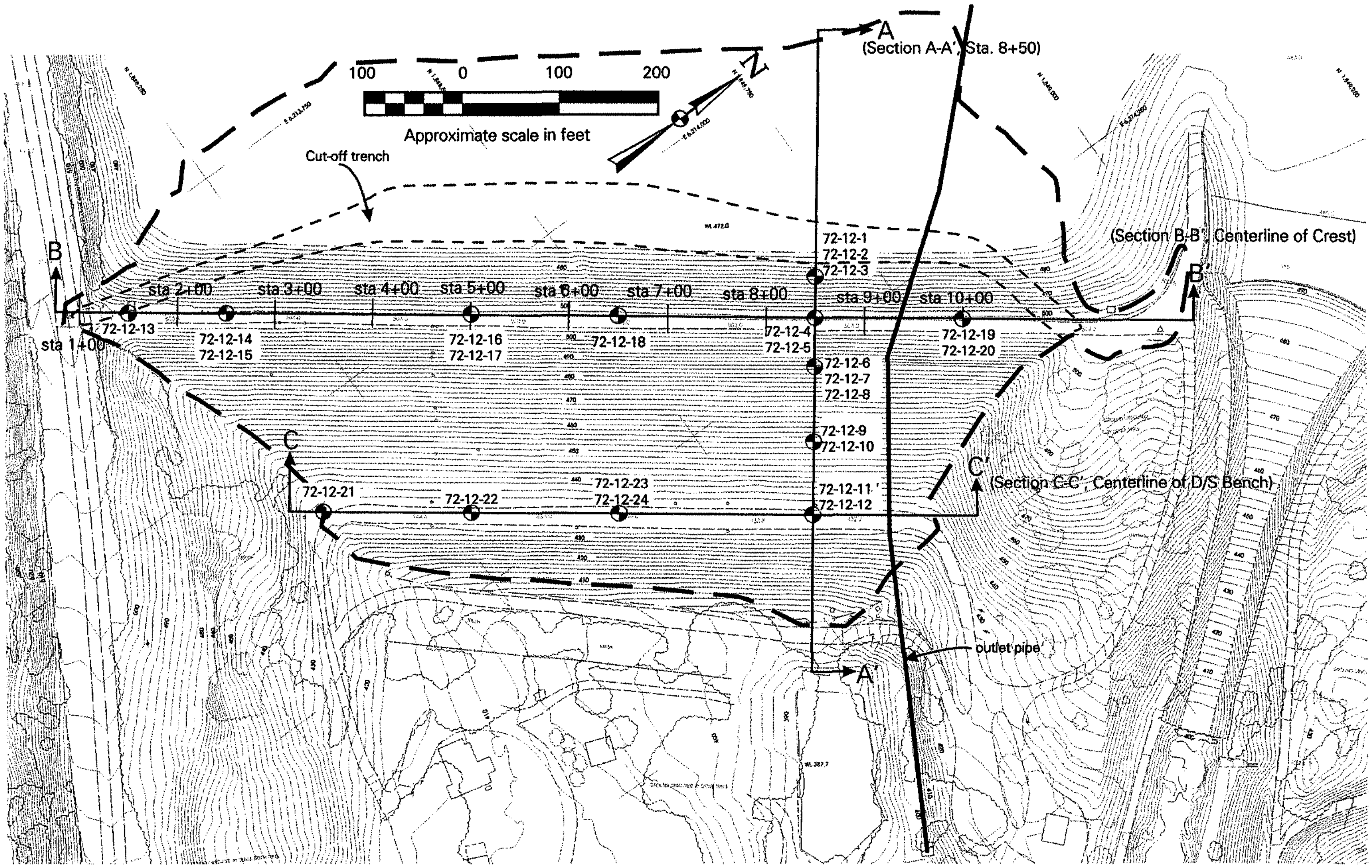
DATE
01/16/02
DESIGN
RLV
DRAWN
JLN
CHECKED
RLV

Santa Clara Valley Water District

New Piezometer Locations

Stevens Creek Dam (No. 72-7)-Sections

SCALE
1″=100′
FIGURE
H-17



Notes:
1) file W:\geolgy\dam safety program\dam safety rport\figures\H-18.cdr
2) basis of elevations is NAVD 1988-elevations are
2.79 feet higher than NGVD 1929

DATE	01/27/02
DESIGN	RLV
DRAWN	JLN
CHECKED	RLV

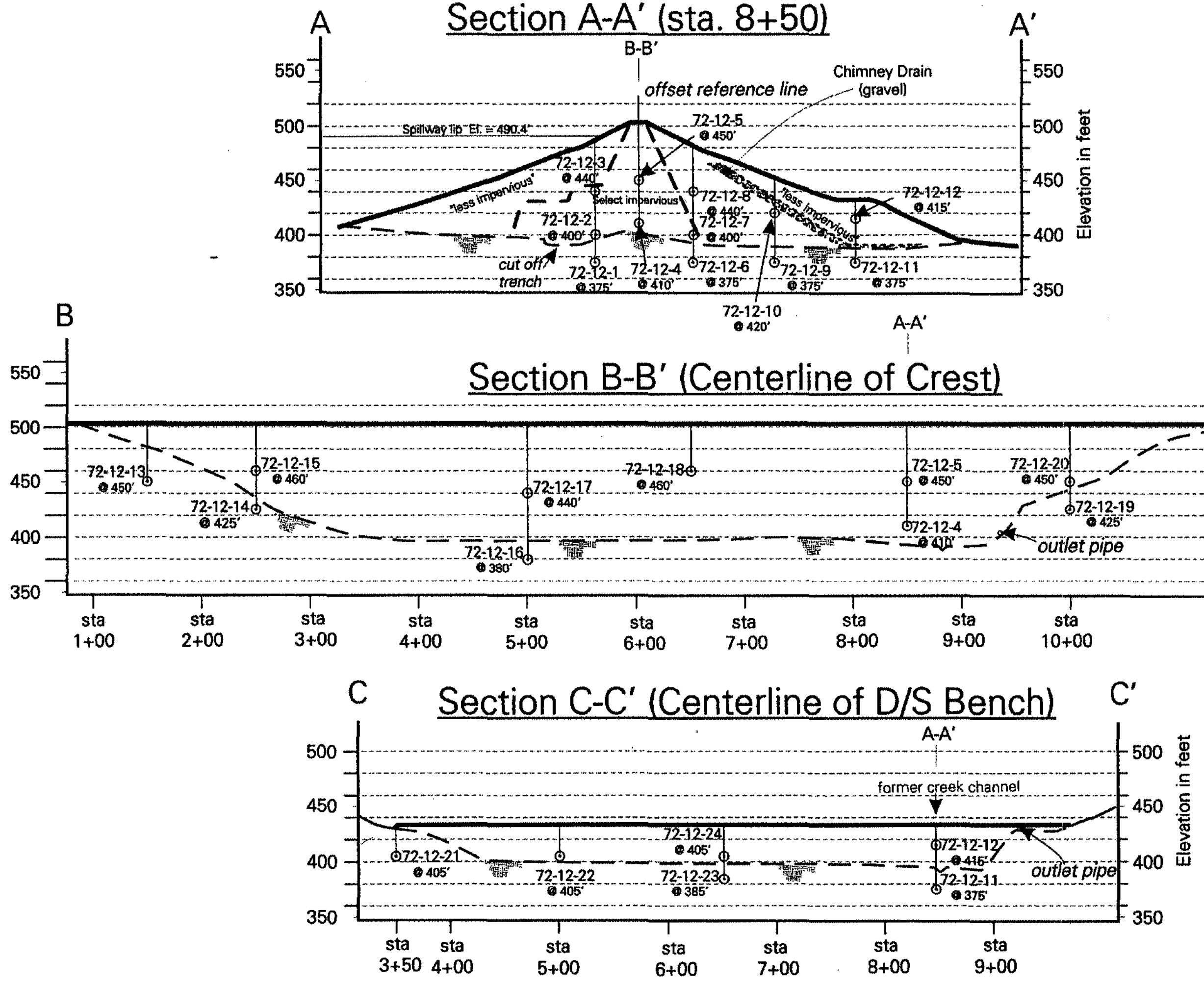
Santa Clara Valley Water District

New Piezometer Locations

Uvas Dam (No. 72-12)-Plan

SCALE
1" = 100'

FIGURE
H-18



Section A-A′ (sta. 8+50)

Section B-B′ (Centerline of Crest)

Section C-C′ (Centerline of D/S Bench)

Notes:
1) file W:\geolgy\dam safety program\dam safety rport\figures\H-19.cdr
2) basis of elevations is NAVD 1988-elevations are
2.79 feet higher than NGVD 1929

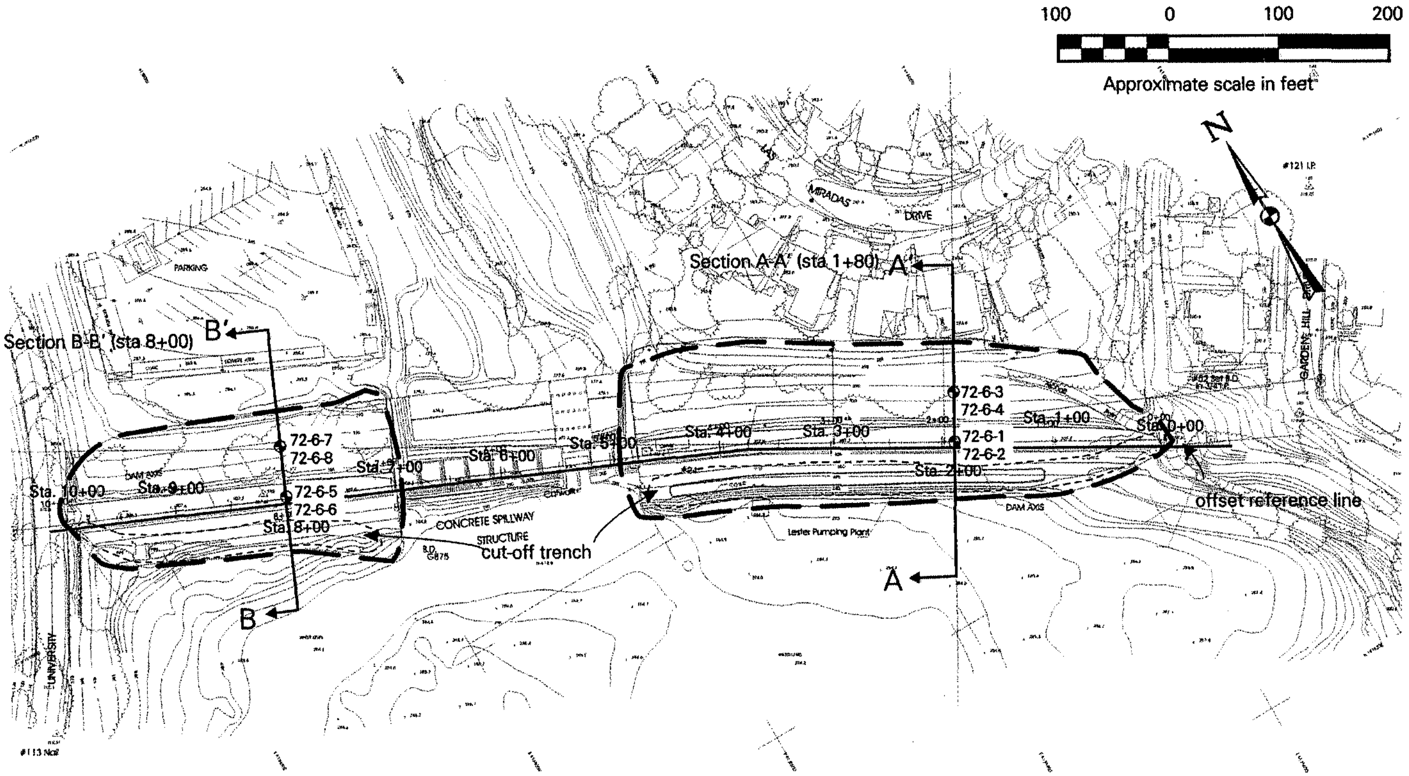
DATE 01/27/02
DESIGN RLV
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

New Piezometer Locations

Uvas Dam (No. 72-12)-Sections

SCALE 1" = 100'
FIGURE H-19



Notes:
1) file W:\geolgy\dam safety program\dam safety rport\figures\H-20.cdr
2) basis of elevations is NAVD 1988-elevations are
2.56 feet higher than NGVD 1929

DATE
01/27/02
DESIGN
RLV
DRAWN
JLN
CHECKED
RLV

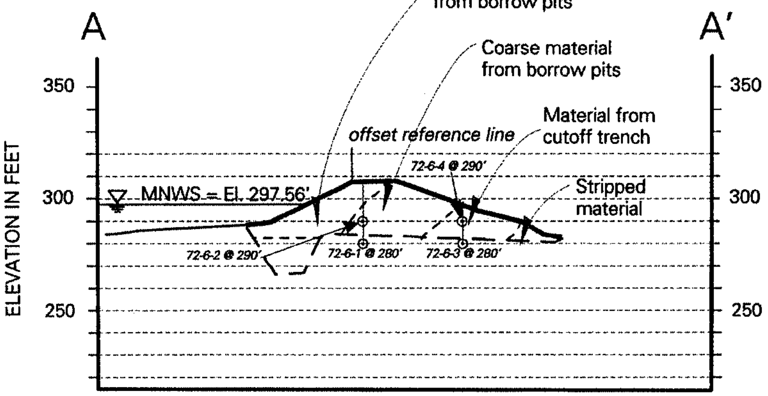
Santa Clara Valley Water District

New Piezometer Locations

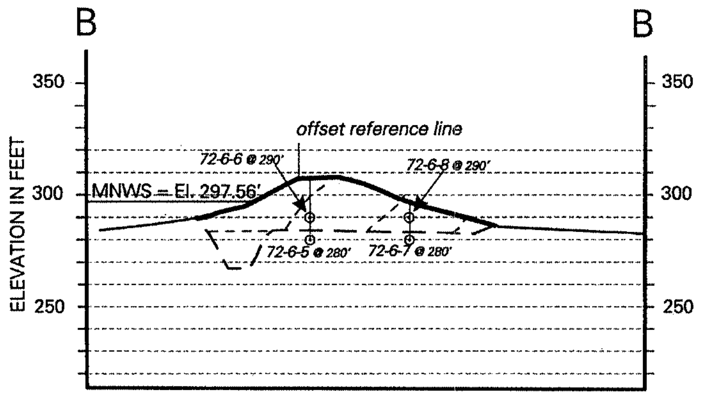
Vasona Dam (No. 72-6)-Plan

SCALE
1"= 100'
FIGURE
H-20

Section A-A′ (sta. 1+80)



Section B-B′ (sta. 8+00)



Notes:
1) file W:\geolgy\dam safety program\dam safety rport\figures\H-21.cdr
2) basis of elevations is NAVD 1988-elevations are
2.56 feet higher than NGVD 1929

DATE 01/27/02
DESIGN RLV
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

New Piezometer Locations

Vasona Dam (No. 72-6)-Sections

SCALE 1"= 50′
FIGURE H-21

APPENDIX I

PERTINENT TECHNICAL REFERENCES FOR DISTRICT DAMS

Appendix I

Technical References Related To District Dams

This appendix presents list of general and technical references for all District dams. The references are presented first by a general list of references that are considered applicable to all dams, which is then followed by specific references for each individual dam, in alphabetical order for each dam. Many of the general references are for geologic articles that current technical staff has reviewed for various aspects of the Dam Safety Program. Except for some of the General References, all references are contained in our District files. Correspondence is specifically excluded from the list of references.

It should be noted that the District files are not complete when it comes to its dams. In the mid-1990s, an unfortunate decision was made to purge "older" files from the District library and records management groups. Fortunately, the error of this management decision was rectified, but not before many original pictures, video tapes, maps and other documents were purged. As a result, in searching for historical records in the future, it is strongly recommended that DSOD files in Sacramento be reviewed.

The District is currently compiling a searchable (computer aided) photographic file retrieval system for all photographs related to the construction of our dams. The system includes about 6,000 images and hopefully will be available by 2004.

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